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SWEETWATER RIVER CHANNEL IMPROVEMENT PROJECT SAN DIEGO
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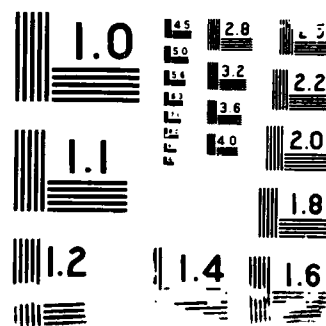
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SWEETWATER RIVER CHANNEL IMPROVEMENT PROJECT SAN DIEGO COUNTY, CALIFORNIA

Hydraulic Model Investigation

by

Herman O. Turner, Jr.

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631



February 1988

Final Report

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<p>The Sweetwater River is located in San Diego County, California. In the area of proposed improvement, the Sweetwater River is a poorly defined channel varying from 1,200 to 2,000 ft wide in a relatively broad floodplain. An entrenched trapezoidal channel with a base width of 320 ft has been excavated ending just upstream of a freeway bridge. This channel has a radius of 1,000 ft and turns approximately 80 deg in relation to the proposed channel alignment through the freeway bridge. A drop structure is to be located in the radius of the curve at the beginning of the project.</p> <p>A study of the proposed project was conducted using a fixed-bed model constructed at a scale of 1:40 to study the effect of downstream waves and disturbances caused by the curvilinear flow conditions. The main objectives of the model study were to obtain quantitative information on flow patterns, flow distribution, waves, and disturbances</p> <p style="text-align: right;">(Continued)</p>					
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throughout the curved reach of channel, as well as to determine the effects of sediment buildup on water-surface elevations.

Testing of the project began with the upstream approach and proceeded downstream to the other areas. The model study revealed that certain refinements are needed to the Sweetwater River project to eliminate potential problems.

The upstream approach as designed created an uneven distribution of flow which affected the energy dissipation of the drop structure. A levee installed on the left overbank area resulted in improved upstream flow conditions at discharges which would ordinarily have created flow over the left bank area.

The slope of the grouted stone drop structure should be changed from 1V on 3H to 1V on 2H. The testing program revealed that better energy dissipation and downstream flow conditions will result if the steeper slope is used.

The side channel overflow spillway did not create any adverse flow conditions. This was due to low discharges coming over the spillway relative to the much larger discharge which would occur simultaneously in the Sweetwater River.

Whenever the natural deposition of bed material forms the sediment buildup in the main channel, higher water-surface elevations will result. These higher water-surface elevations will cause flooding along the right side of the main channel in the overflow spillway and bridge pier areas. Higher levee heights are recommended for these areas.

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PREFACE

The model investigation reported herein was authorized by Headquarters, US Army Corps of Engineers (USACE), on 20 September 1984 at the request of the US Army Engineer District, Los Angeles (SPL), through the US Army Engineer Division, South Pacific (SPD). The model tests were accomplished during the period December 1984 to June 1985 in the Hydraulics Laboratory (HL) of the US Army Engineer Waterways Experiment Station (WES) under the general supervision of Messrs. F. A. Herrmann, Jr., Chief, HL, and J. L. Grace, Jr., Chief, Hydraulic Structures Division (HSD), and under the direct supervision of Messrs. G. A. Pickering, HSD, and J. G. George, Chief, Locks and Conduits Branch (LCB). The tests were conducted by Messrs. H. O. Turner, Jr., and J. E. Myrick, LCB. This report was prepared by Mr. Turner and edited by Mrs. Marsha C. Gay, Information Technology Laboratory.

Messrs. Tom Munsey of USACE, Dick DiBuono of SPD, and Joe Evelyn and Eddie Chew of SPL visited WES during the course of the model study to observe model operation and correlate results with design studies.

COL Dwayne G. Lee, CE, is the Commander and Director of WES.
Dr. Robert W. Whalin is the Technical Director.



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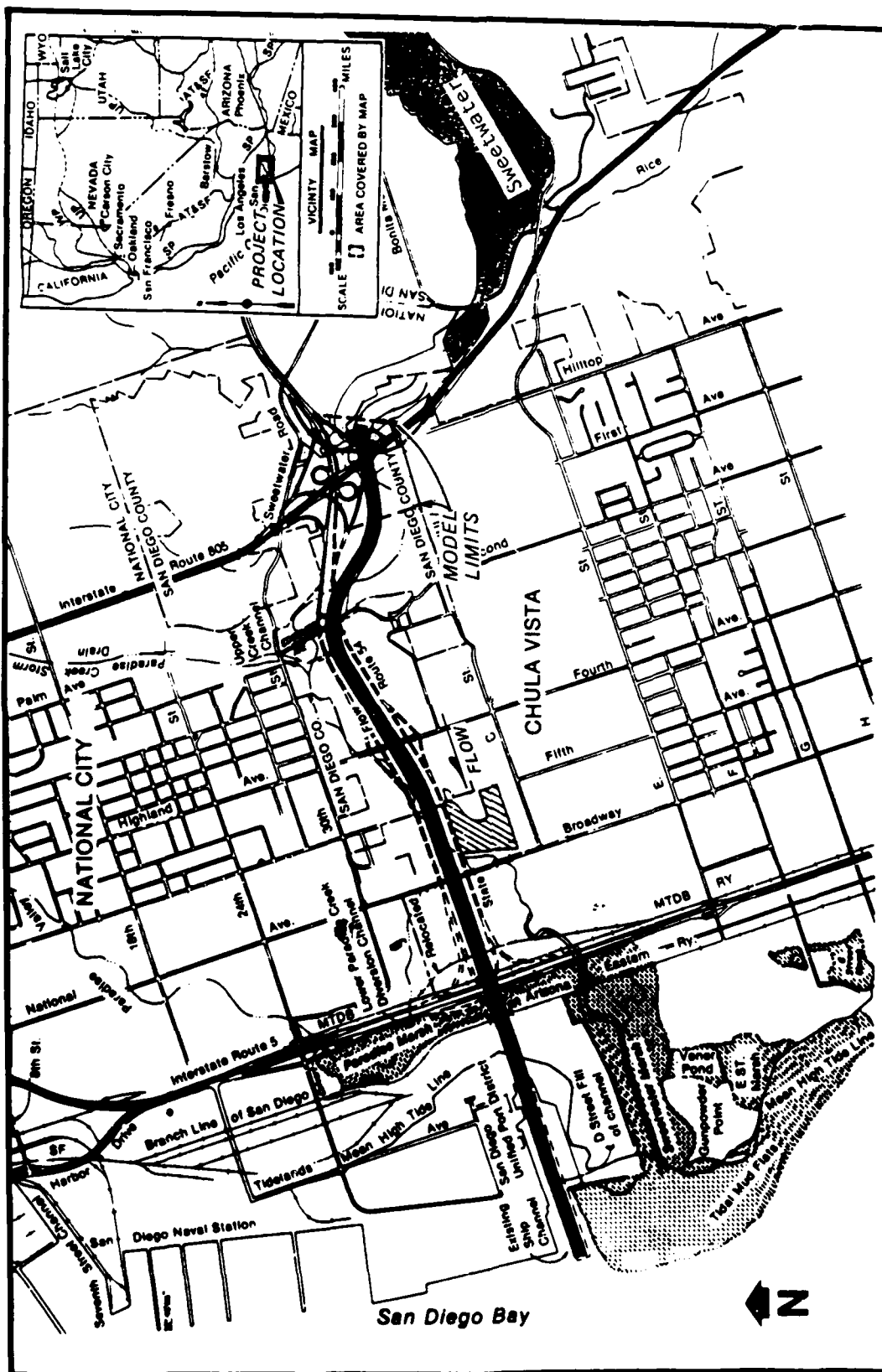
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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI
(metric) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
cubic feet	0.0283168	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
miles (US statute)	1.609347	kilometres



SWEETWATER RIVER CHANNEL IMPROVEMENT PROJECT

SAN DIEGO COUNTY, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Sweetwater River is located in San Diego County, California (Figure 1). In the area of proposed improvement, the Sweetwater River is a poorly defined channel varying from about 1,200 to 2,000 ft* wide in a relatively broad floodplain. A major change has been made in the drainage pattern since this project was authorized. The lower end of Lower Paradise Creek was filled and a diversion channel excavated to the existing Sweetwater River. This diversion channel is located between the San Diego and Arizona Eastern Railway branch line and Interstate Route 5. Local interests have excavated an entrenched trapezoidal channel with a base width of 320 ft to just upstream of the Interstate 805 Freeway. This channel has a radius of 1,000 ft and turns approximately 80 deg in relation to the proposed channel alignment through the freeway (Plate 1). A drop structure is to be located in the radius of the curve at the beginning of the project. The Standard Project Flood (SPF) is 60,000 cfs.

Purpose of Model Investigation

2. A model study of the proposed project was conducted to study the effects of downstream waves and disturbances caused by the curvilinear flow conditions. These flow conditions may cause water to escape through the side drain overflow spillway on the right bank of the curve. The main objectives of the model study were to obtain quantitative information on flow patterns, flow distribution, waves, and disturbances throughout the curved reach of channel; to determine the effects of sediment buildup on water-surface

* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

elevations in this reach of channel; and to develop corrective procedures where problems are defined.

Scope

3. The scope of the model investigation involved studying the hydraulic problems created by locating a drop structure in the bend of a channel. The problems were further complicated by a side channel overflow spillway located downstream of the drop structure and bridge piers. Once a suitable plan was developed, the predicted sediment deposition was installed and studied.

PART II: THE MODEL

Description

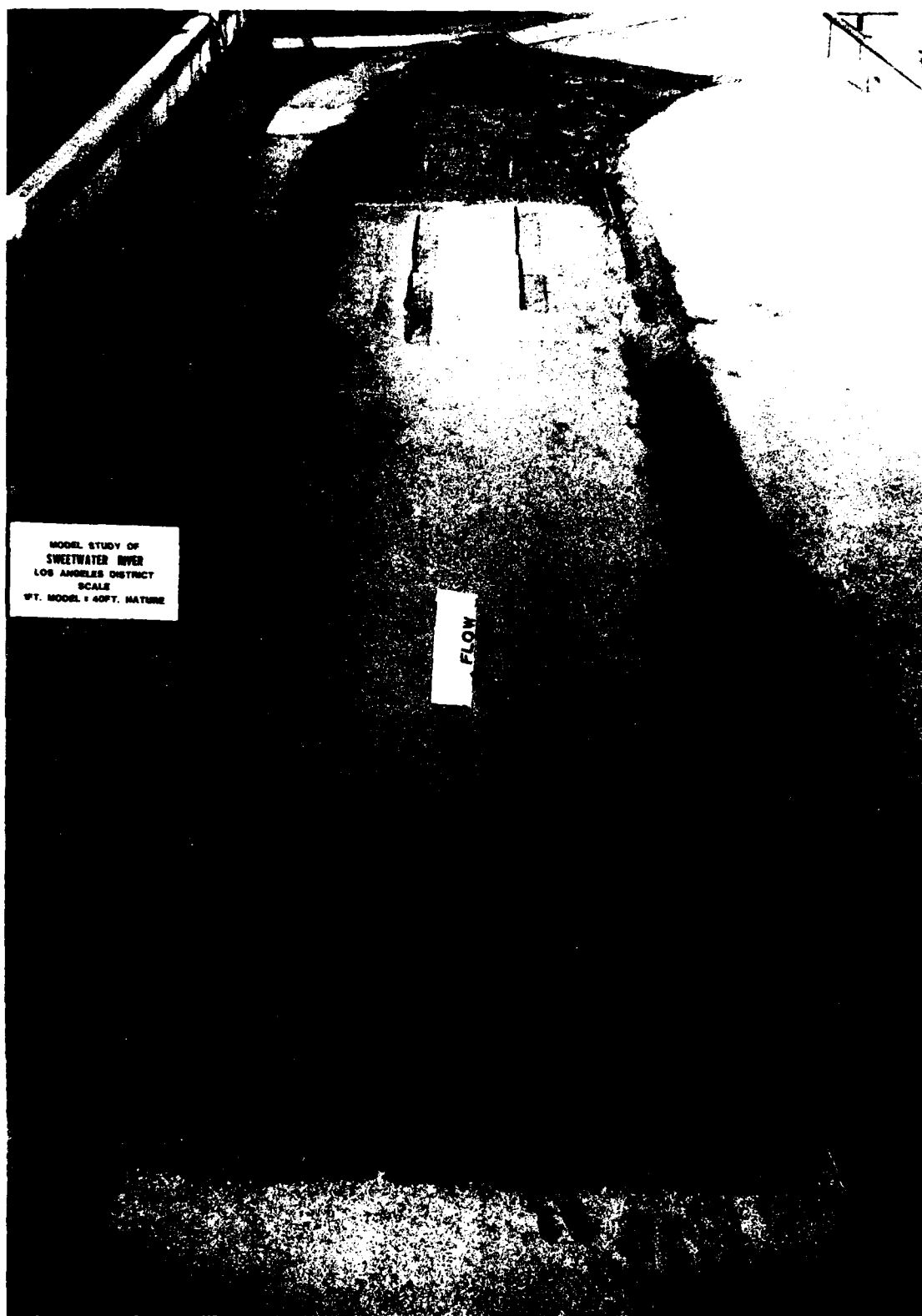
4. The investigation was conducted using a 1:40-scale model (Figure 2). The model reproduced approximately 3,300 ft of trapezoidal channel from sta 193+00 to sta 160+00 that included the drop structure and 760 ft of curved approach channel to the drop structure. From sta 193+00 to sta 185+40, the channel was 320 ft wide. The channel then transitioned from a width of 320 ft to 250 ft between sta 185+40 and sta 181+45. From sta 181+45 to sta 160+00 the channel was 250 ft wide. Portions of the channel were molded in sand with scaled riprap placed on filter cloth on the side slopes. The approach channel was molded in cement mortar to sheet metal templates. A scaled mixture of crushed stone, mixed according to prototype gradations, was placed in the appropriate areas to accurately simulate the prototype riprap. In the areas which required grouted stone, cement dust was sprinkled over the crushed stone, moistened, and allowed to harden.

Model Appurtenances

5. Water used in the operation of the model was supplied by a circulating system. Discharges for the main channel were measured with venturi meters. The side channel overflow spillway discharges were measured with an electronic flowmeter. Steel rails graded to specific elevations were placed along both sides of the model to serve as supports for measuring devices and to provide a convenient means of establishing stations and elevations in the model. Velocities were measured with either a pitot tube or an electronic velocity meter. Tailwater elevations were regulated by an adjustable gate at the end of the flume. Water-surface elevations were measured with point gages. Different designs along with different flow conditions were recorded photographically.

Scale Relations

6. The equations of hydraulic similitude, based on Froudian relations, were used to express mathematical relations between the dimensions and



MODEL STUDY OF
SWEETWATER RIVER
LOS ANGELES DISTRICT
SCALE
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FLOW

Figure 2. General view of model

hydraulic quantities of the model and prototype. General relations for transferring model data to prototype equivalents are as follows:

<u>Characteristic</u>	<u>Dimension*</u>	<u>Model:Prototype Scale Relations</u>
Length	L_r	1:40
Area	$A_r = L_r^2$	1:1,600
Velocity	$V_r = L_r^{1/2}$	1:6.3246
Discharge	$Q_r = L_r^{5/2}$	1:10,119
Time	$T_r = L_r^{1/2}$	1:6.3246

* Dimensions are in terms of length.

PART III: TESTS AND RESULTS

7. Initial tests were conducted to observe general flow conditions with various discharges Q up to the SPF (60,000 cfs), and to determine the adequacy of various aspects of the project. Water-surface elevations, velocities, and photographs of the model were obtained. Water-surface profiles with discharges of 21,000, 45,000, and 60,000 cfs are shown on Plates 2-4, respectively. Also shown in these plates are profiles from HEC-2 computer outputs furnished by the US Army Engineer District, Los Angeles.*

8. Problems were identified and modifications made beginning at the upstream end of the test section and moving downstream, since each modification could affect flow conditions in the downstream sections of the model. Various areas of the model, beginning at the upstream end, are shown in Photos 1-3. Tests conducted in these areas are discussed in subsequent paragraphs.

Approach Channel

Original design

9. The superelevated upstream approach to the drop structure (Photo 1) begins at sta 193+00. This station represents the point of curvature (PC) of the curved channel, which has a radius of 1,000 ft and turns approximately 80 deg. The curved portion of the channel (Photo 4) ends at the point of tangency, sta 179+50. A grouted stone drop structure at sta 185+67.90 has a superelevation slope of 0.004688 ft/ft along the crest. The difference in elevation between the right and left sides is 1.5 ft (Plate 5). All superelevation ends at the toe of the drop structure.

10. Flow conditions in the upstream approach channel are shown in Photo 5 for discharges of 21,000, 45,000, and 60,000 cfs. Dye added at the toe of the inside bank at sta 193+00 appears in the flow as a dark streak along the inside bank. Confetti was added to the model to observe the flow patterns in the approach channel. These confetti patterns for the same discharges are shown in Photo 6. As the dye streaks and confetti patterns show for discharges of 45,000 and 60,000 cfs, the flow pattern is pushed away from

* US Army Engineer District, Los Angeles. 1984 (Apr). "Interior Drainage Supplemental Report; Sweetwater River, California, San Diego County," Los Angeles, Calif.

the inside bank creating an uneven flow distribution at the drop structure. This flow pattern was caused by the flow overtopping the left overbank and spilling back into the approach channel (Photos 5b and c). Any water on the left bank caused by the higher discharges intersected the main channel flow and caused an uneven distribution of flow just upstream of the drop structure. Differences in water-surface elevations between the right and left banks are shown in Plate 6. As a result, an uneven hydraulic jump was formed at the drop structure. Velocity patterns in the upstream approach channel (Plates 7-9) show the magnitude and direction of velocities for the type 1 design with a discharge of 60,000 cfs.

Type 2 design

11. A levee was placed on the left upstream bank (type 2 design approach channel) in an attempt to improve the flow distribution in the approach channel (Figure 3). This levee was formed by extending the left side slope to approximately the same elevation as the right side of the channel. The levee began at sta 193+00 and joined an existing levee at sta 188+00. The purpose of the levee was to prevent any flow from entering the trapezoidal channel from the left overbank area. Flow conditions were observed with the type 2 design approach channel using dye streaks (Photo 7) and confetti (Photo 8) with discharges of 21,000, 45,000, and 60,000 cfs. Test results indicated that approach conditions to the drop structure were significantly improved with the type 2 design (Photos 7b and 7c) compared with the original design (Photos 5b and 5c). Improved flow conditions in the approach channel resulted in more uniform flow conditions in the grouted stone drop structure for the discharges observed. Water-surface elevations for the type 2 upstream approach are shown in Plate 10. For lower discharges of 31,000 cfs and less where flow does not overtop the left bank, the levee had no effect on flow conditions in the approach channel. Velocity patterns in the upstream approach channel (Plates 11-13) show the magnitude and direction of velocities for the type 2 design approach curve with a discharge of 60,000 cfs.

Grouted Stone Drop Structure

Original design

12. A grouted stone drop structure with a 1V on 3H slope (Photo 4) is located at sta 185+67.90. The crest of the drop structure is superelevated to match the superelevation of the approach channel. Elevations of the right and



Figure 3. Location of levee on left bank of approach channel to grouted drop structure, type 2 design approach channel

left sides are 22.0 and 20.5 ft, respectively. The toe of the drop structure (sta 185+40) has an elevation of 11.95 ft. All superelevation of the upstream approach channel ends at the toe of the drop structure. Details of the type 1 drop structure are shown in Plate 14.

13. Flow conditions were observed in the drop structure with both the type 1 (original) and type 2 design approach channels. With a discharge of 21,000 cfs, an undular type hydraulic jump occurred in the drop structure causing secondary waves to form in the channel (Photo 9). This jump is considered unstable and oscillates between a plunging jet and a jet that rides the surface. Velocities taken downstream of the drop structure show the result of the undular jump at 21,000 cfs (Plate 15). With discharges above 31,000 cfs, flow entered the approach channel from the left bank just upstream of the drop structure with the type 1 design approach channel (Photos 5b and c). This caused unsymmetrical flow conditions at the drop structure, reducing the effectiveness of the hydraulic performance of the drop structure. Velocities taken downstream of the drop structure with types 1 and 2 approach channels at a discharge of 60,000 cfs are shown in Plate 16. With the type 2 design approach channel, approach conditions were significantly improved, resulting in more uniform flow conditions in the drop structure. However, even with the improved approach conditions, the drop structure with the 1V on 3H slope did not perform satisfactorily for all discharges observed.

Type 2 design drop structure

14. The slope of the drop structure was changed from 1V on 3H to 1V on 2H (Photo 10) in an attempt to improve the hydraulic performance of the drop structure for the full range of discharges observed. This was designated the type 2 design drop structure (Plate 17). As discussed previously, flow conditions observed with the original approach conditions were unsatisfactory in the drop structure because the flow just upstream of the drop structure was not uniformly distributed across the width of the channel for the higher flows. Plate 18 shows the downstream velocities with the types 1 and 2 approach channels and the type 2 drop structure. With the levee installed (type 2 design approach channel) in conjunction with the type 2 design drop structure, flow conditions were satisfactory for the discharges observed because the flow was uniformly distributed across the approach channel upstream of the drop structure, as shown in Photos 7 and 8. This produced a stable, more efficient hydraulic jump in the drop structure for all discharges

observed, and therefore is the recommended design.

Side Channel Overflow Spillway

15. A major concern of the model study was to determine the effect of flow from the side channel spillway on flow conditions in the main channel. The side channel overflow spillway (Photo 2), located on the right side of the channel between sta 184+00 and 180+85, discharges flow into the main channel immediately downstream of the grouted stone drop structure. Flow rates were specified according to information provided by the Los Angeles District.* This information, proved in the following tabulation, lists the side channel spillway flow as a function of the main channel flow.

<u>Side Channel Spillway Flow, cfs</u>	<u>Sweetwater River Flow, cfs</u>
1,490 (100 years)	4,500 (25 years)
4,300 (SPF)	21,000 (50 years)
57	60,000 (SPF)

16. Tests conducted with the SPF of 4,300 cfs entering the side channel with a discharge of 21,000 cfs in the main channel (Photo 11) indicated the flow from the overflow spillway had little effect on flow conditions downstream of the drop structure. For comparison, flow conditions with a discharge of 21,000 cfs in the main channel and no flow entering from the overflow spillway are shown in Photo 12. Although not specified as a probable condition, the SPF discharges in both the main channel and overflow spillway were observed, as shown in Photo 13. No adverse flow conditions were caused by the side channel overflow spillway under any of the conditions observed; therefore, no modifications were recommended.

Bridge Piers

17. The bridge piers, shown in Photo 3, are located approximately 500 ft downstream from the drop structure. In the model, the tops of the

* Personal Communication, "Sweetwater River General Design Memorandum; Transmittal of Project Data for Hydraulic Model Study," 18 September 1984, US Army Engineer District, Los Angeles, Los Angeles, Calif.

piers were set at el 30.5* (well above the water surface) although the prototype elevations would be much higher. Each pier was expected to collect up to 4 ft of channel debris on its nose, according to information obtained from the Los Angeles District.** The debris was simulated in the model by attaching loosely woven fibrous material to each pier as shown in Photo 14.

18. Standing waves created by the bridge piers were present for the various discharges observed (Photo 15). The interaction of these standing waves did cause an increase in the water-surface elevation at certain locations in this reach of channel. However, due to the spacing of the piers and the elevations of the bridges, the standing waves did not present a problem. Plate 19 shows the position and location of the bridge piers. Maximum water-surface elevations recorded at a discharge of 60,000 cfs are shown in Plates 20-23.

Sediment Wedge

19. A sediment transport analysis performed by the Los Angeles District** indicated that due to the gradient of the upstream channel (0.0030 ft/ft) being substantially greater than that of the improved channel (0.0010 ft/ft), deposition of sediment would likely occur downstream from the inlet drop structure during the passage of a major flood. This analysis indicated that a sediment wedge would develop from the grouted rock stabilizer at sta 130+00 to the drop structure at sta 185+67.90. The maximum depth of the wedge would be about 4 ft just downstream of the drop structure tapering down to no deposition at sta 130+00. The possible effects of the sediment deposition would be greater channel velocities and higher water-surface elevations, which may require revised levee heights.

20. To evaluate the effects of the expected sediment buildup, sand was added to the bed of the model and molded until the proper elevations were reached. Then the molded sand was sprinkled with a layer of cement dust which was allowed to harden. This procedure produced a fixed sediment bed as shown in Figure 4.

21. Water-surface elevations were recorded for the various flow

* All elevations (el) and stages cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

** Personal Communication, op. cit.

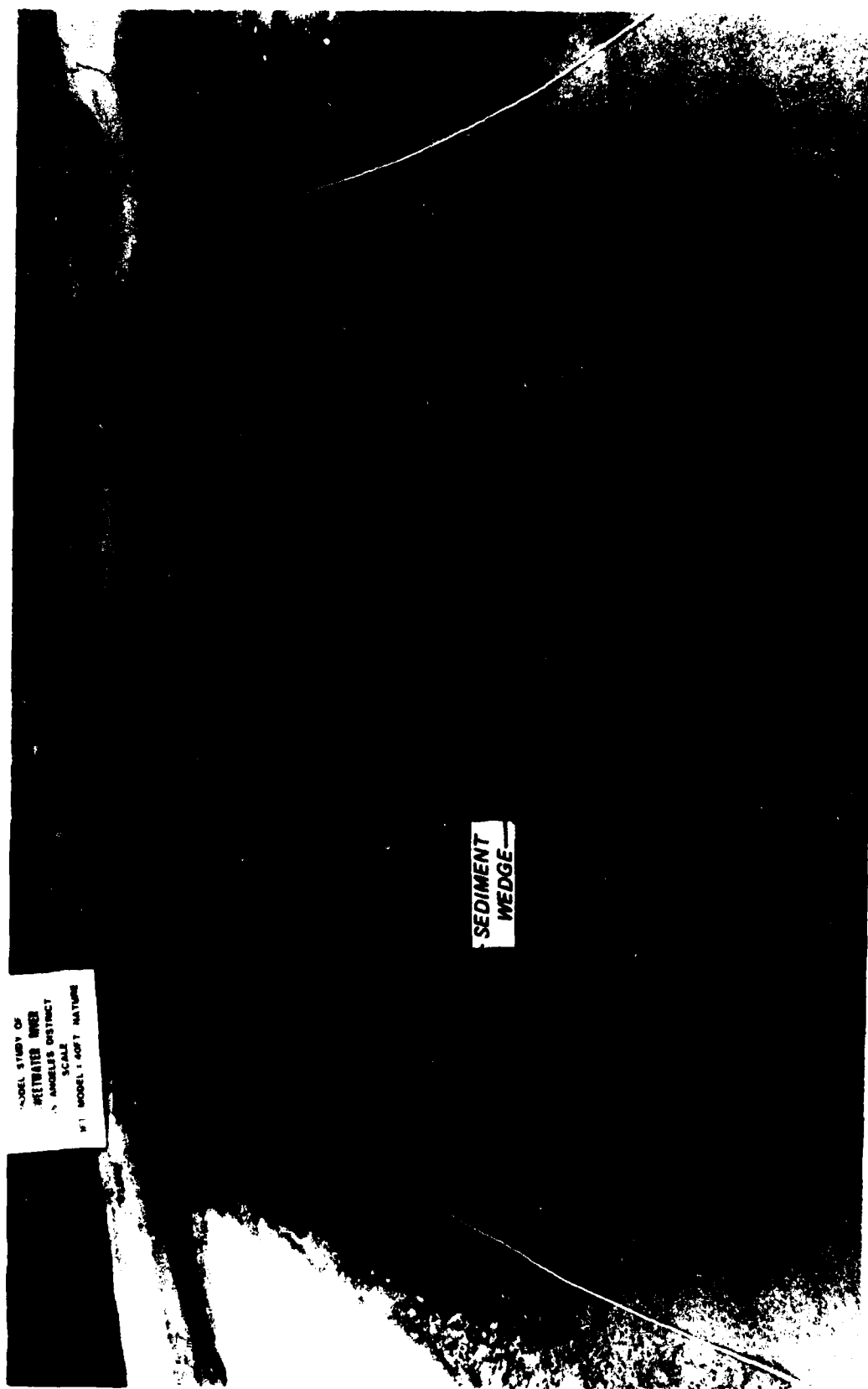


Figure 4. Sediment wedge buildup of bed material downstream of drop structure

conditions. Plate 24 shows the maximum water-surface elevations without the sediment wedge for the design discharge of 60,000 cfs. Water-surface elevations with the sediment wedge installed are shown in Plate 25. Levee heights and bed elevations are also plotted in Plates 24 and 25.

22. Tests conducted with the sediment wedge in place indicated the drop structure was ineffective with the higher discharges. With the design discharge of 60,000 cfs, flow overtopped the overflow spillway and the right bank (Photo 16), indicating additional levee height would be required to contain the flow in the main channel. Different flow conditions with the sediment wedge in place are shown in Photo 17. Water-surface elevations around the bridge piers created by a discharge of 60,000 cfs with the sediment wedge installed are shown in Plates 26-29.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

23. The model study revealed that certain refinements were needed to the Sweetwater River project to eliminate potential problems. Testing began with the upstream approach and proceeded downstream to the other areas.

24. The upstream approach as designed created an uneven distribution of flow at the higher discharges which affected the energy dissipation of the drop structure. For discharges greater than 31,000 cfs, flow overtopped the left bank and entered the main channel just upstream of the drop structure. This pattern resulted in unsatisfactory approach conditions to the drop structure, reducing the effectiveness of the hydraulic performance of the drop structure. A levee installed on the left bank contained the flow in the main channel, which significantly improved approach conditions to the drop structure.

25. With the original design drop structure (1V on 3H slope), the hydraulic jump was unstable with the higher range of discharges. The 1V on 3H slope caused the flow to be in a state of transition between a plunging jet, causing a hydraulic jump to develop, and one that rides the surface. Test results indicated that changing the 1V on 3H slope to a 1V on 2H slope with the levee installed upstream on the left bank produced a stable, more efficient hydraulic jump for the full range of flow conditions observed. Therefore, this design was the recommended design for prototype construction.

26. Flow entering the channel from the side channel overflow spillway just downstream of the drop structure had little effect on flow conditions in the main channel. When the peak discharge occurred on the overflow spillway, a high discharge was also present in the main channel. The flow from the side channel overflow spillway simply was not large enough to adversely affect the larger flow in the main channel.

27. The standing waves created by the bridge piers did not present a problem due to the spacing of the piers and the bridge elevations.

28. Tests conducted with a sediment wedge up to 4 ft deep in the channel caused higher water-surface elevations below the drop structure. With the design discharge of 60,000 cfs, flow overtopped the right bank and the side channel overflow spillway. With the deposition of sediment in the channel, additional levee height would be required downstream of the drop structure to contain the higher water-surface elevations from a SPF.

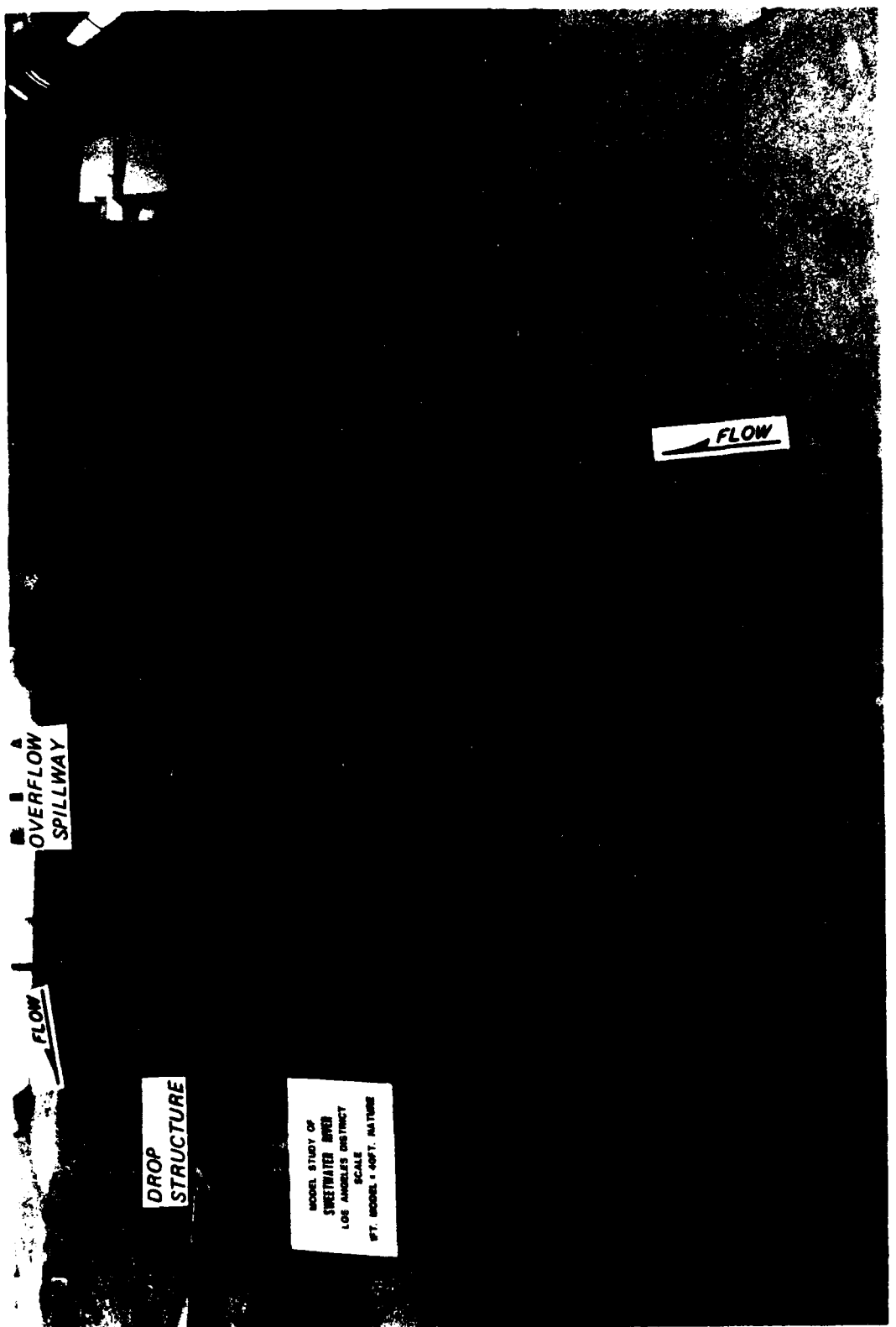


Photo 1. Curved approach channel to grouted drop structure (type 1 design)



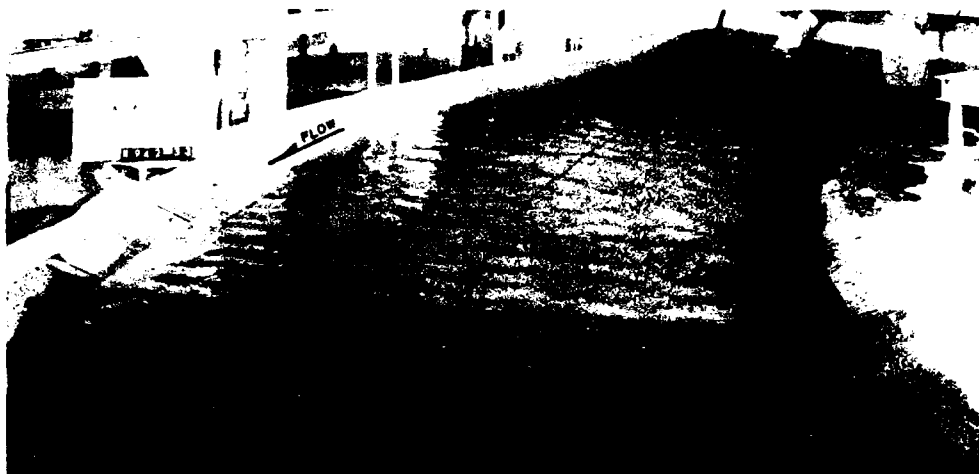
Photo 2. Curved channel with grouted drop structure and overflow spillway (type 1 design)



Photo 3. Portion of trapezoidal channel with bridge piers (type 1 design)



Photo 4. Curved channel with grouted drop structure (type 1 design)



a. $Q = 21,000$ cfs



b. $Q = 45,000$ cfs



c. $Q = 60,000$ cfs

Photo 5. Flow conditions with the type 1 design approach channel to the type 2 design drop structure



a. $Q = 21,000$ cfs



b. $Q = 45,000$ cfs

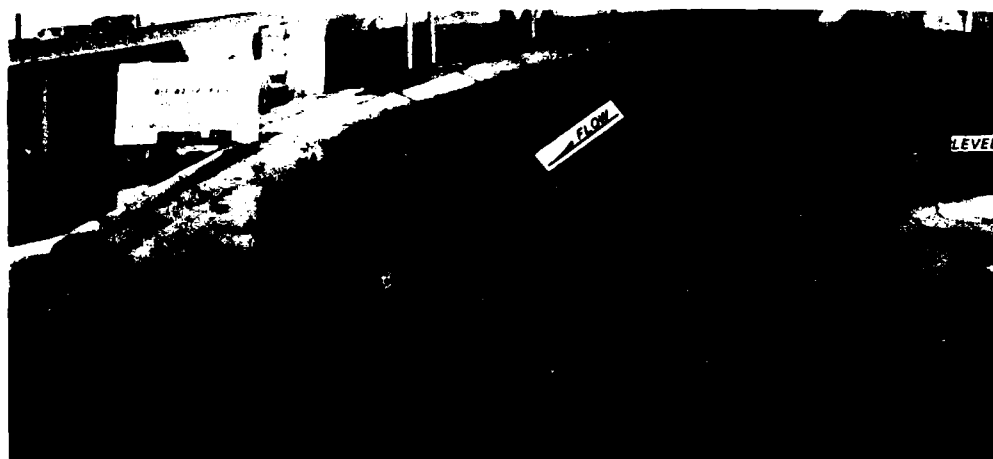


c. $Q = 60,000$ cfs

Photo 6. Flow conditions with the type 1 design approach channel to the type 2 design drop structure. Confetti accents surface flow patterns.
Time exposure 6.3 sec prototype



a. $Q = 21,000$ cfs



b. $Q = 45,000$ cfs



c. $Q = 60,000$ cfs

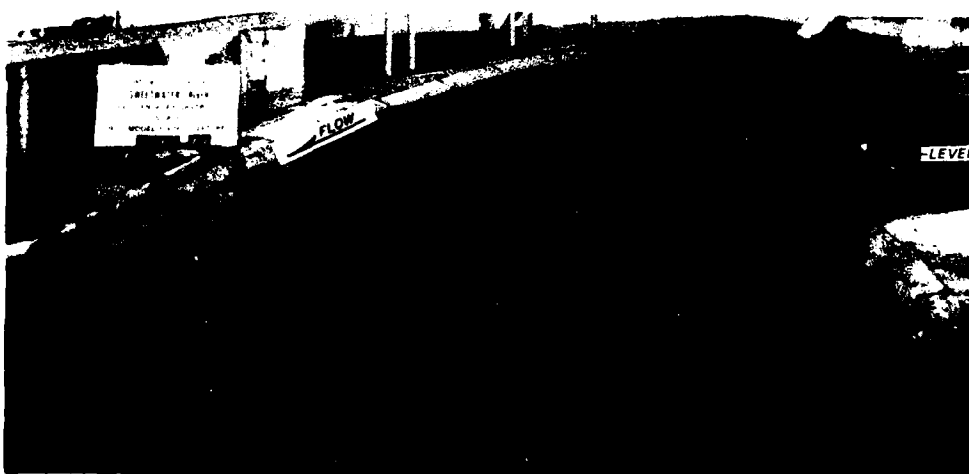
Photo 7. Flow conditions in the type 2 design approach channel with the levee installed and the type 2 design drop structure. Dye illustrates flow patterns along the toe of the left bank



a. $Q = 21,000$ cfs



b. $Q = 45,000$ cfs



c. $Q = 60,000$ cfs

Photo 8. Flow conditions in the type 2 design approach channel and the type 2 design drop structure. Confetti accents surface flow patterns.
Time exposure 6.3 sec prototype



Photo 9. Flow conditions with the type 1 design grouted stone drop structure with a slope of 1V on 3H,
 $Q = 21,000$ cfs

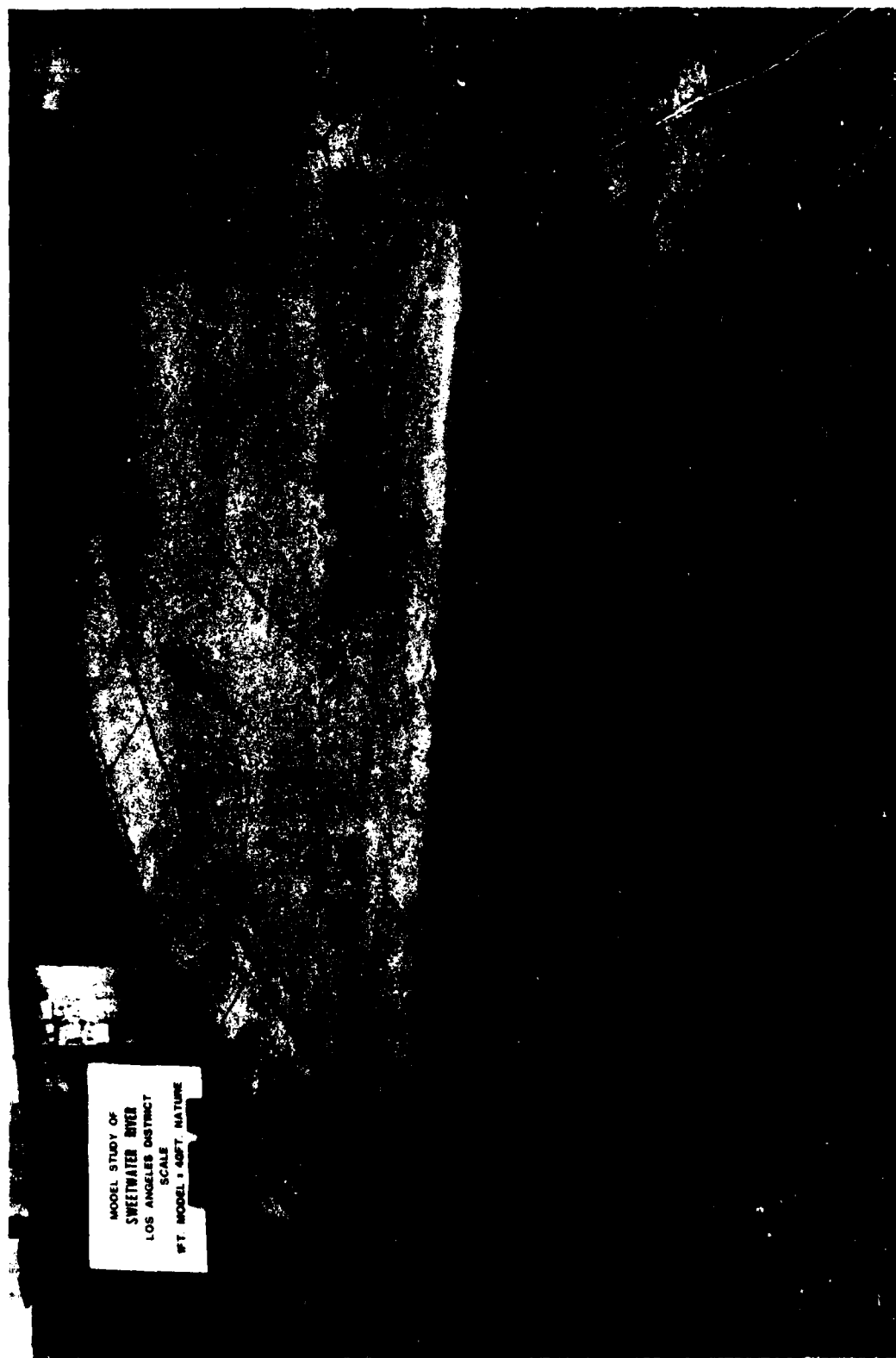


Photo 10. The type 2 design drop structure and type 1 design approach channel



Photo 11. Flow conditions downstream of the drop structure with flow from the overflow spillway entering the main channel. Type 1 design drop structure and type 1 design approach channel. Discharge from Sweetwater River 21,000 cfs; discharge from overflow spillway 4,300 cfs



Photo 12. Flow conditions downstream of the drop structure with no flow entering from the overflow spillway, type 1 design drop structure and approach channel, $Q = 21,000$ cfs

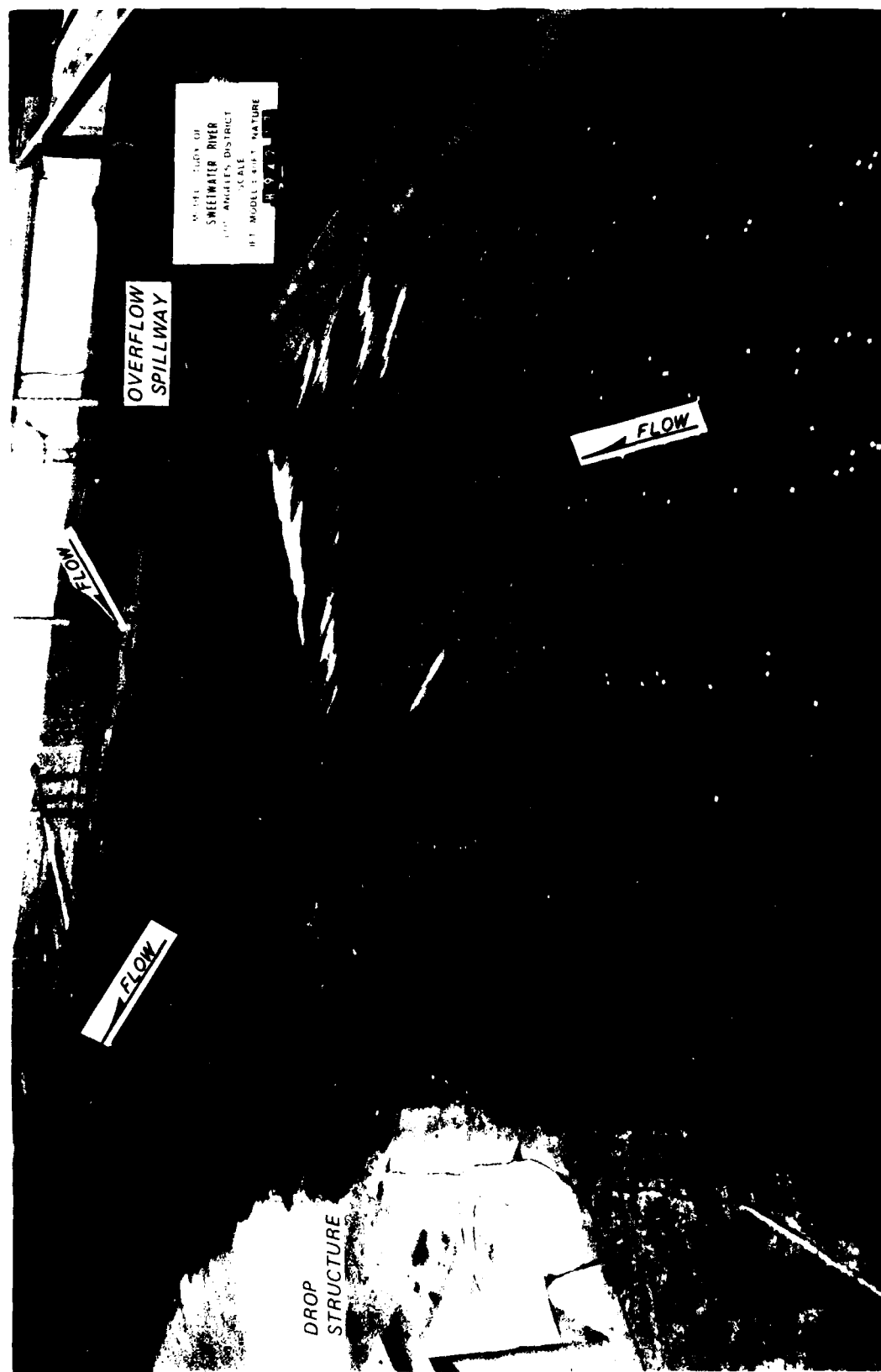


Photo 13. Flow conditions downstream from the drop structure with the SPF in both the main channel and overflow spillway. Type 1 design drop structure and approach channel. Discharge from Sweetwater River 60,000 cfs; discharge from overflow spillway 4,300 cfs



Photo 14. Bridge piers with debris simulated on the nose of each pier



a. $Q = 21,000$ cfs



b. $Q = 45,000$ cfs



c. $Q = 60,000$ cfs

Photo 15. Flow conditions with standing waves present at bridge piers
with debris attached to the pier nose

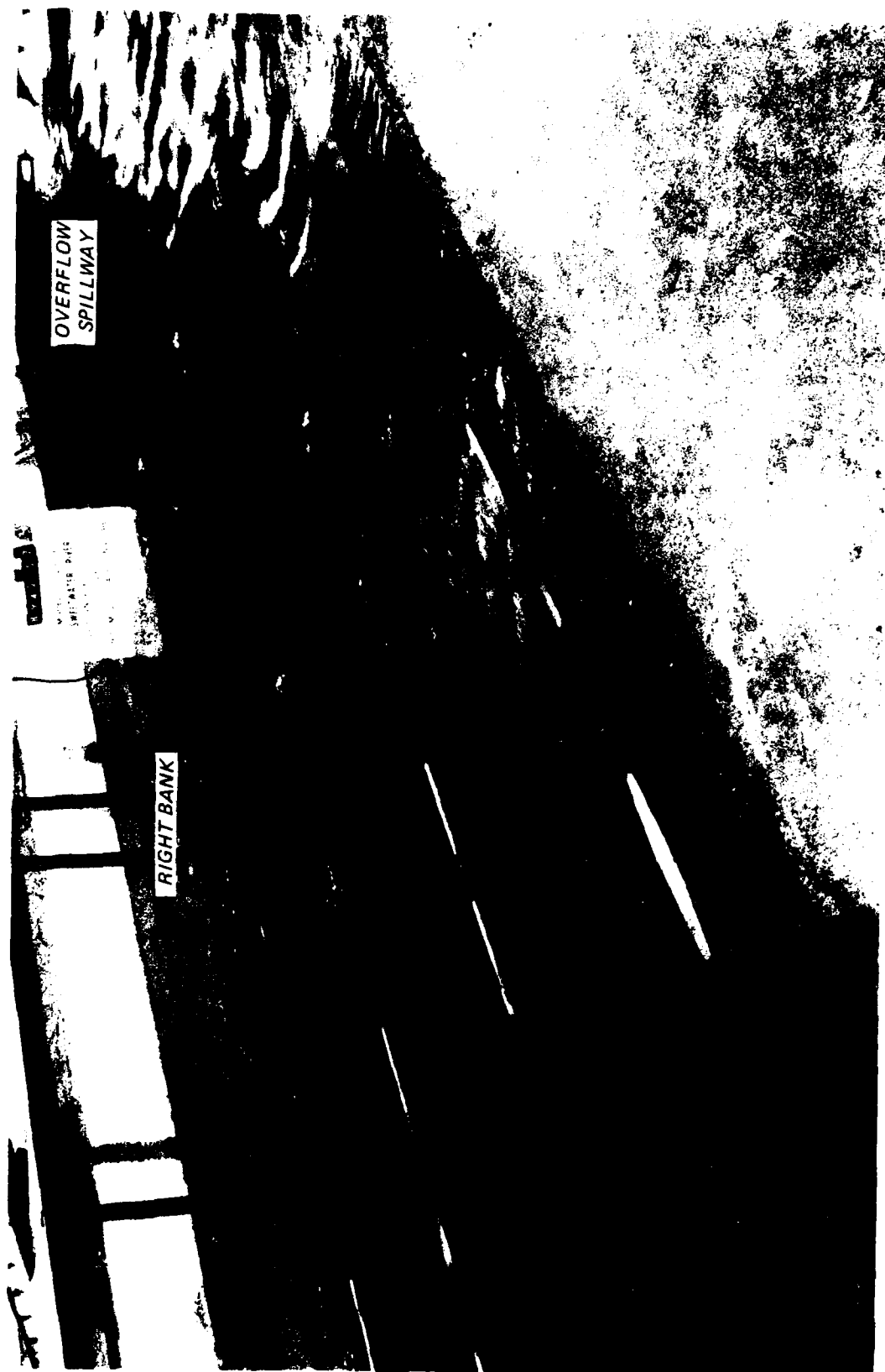
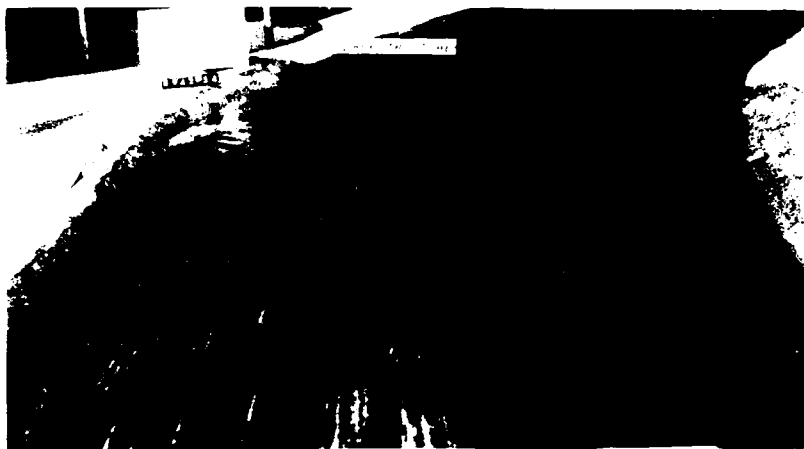
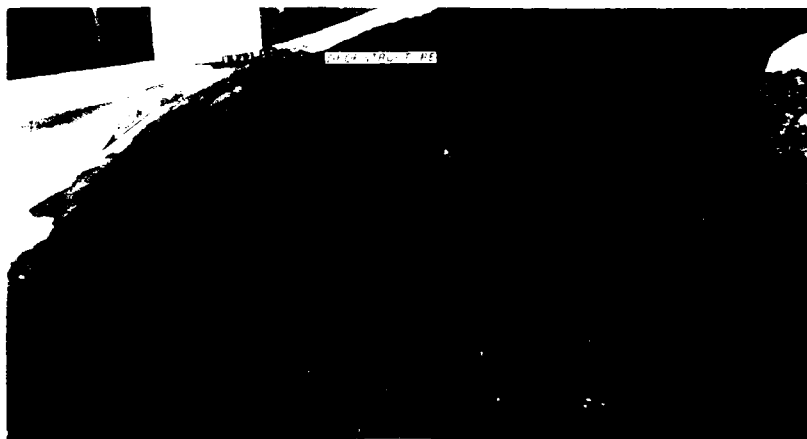


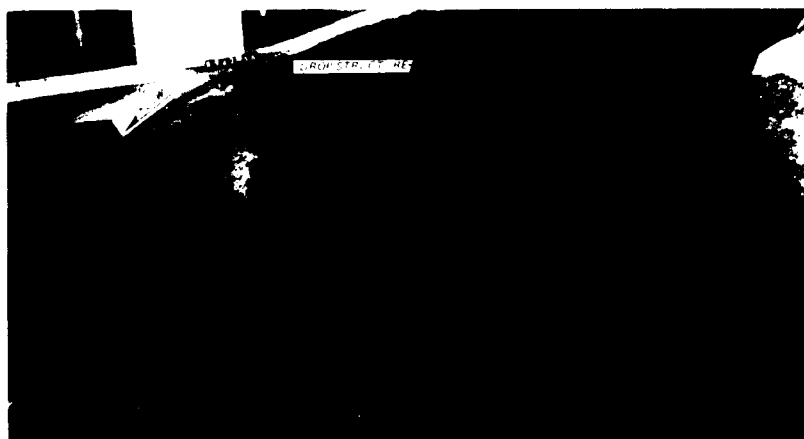
Photo 16. Flow conditions with the design discharge of 60,000 cfs with the sediment wedge in place



a. $Q = 21,000$ cfs



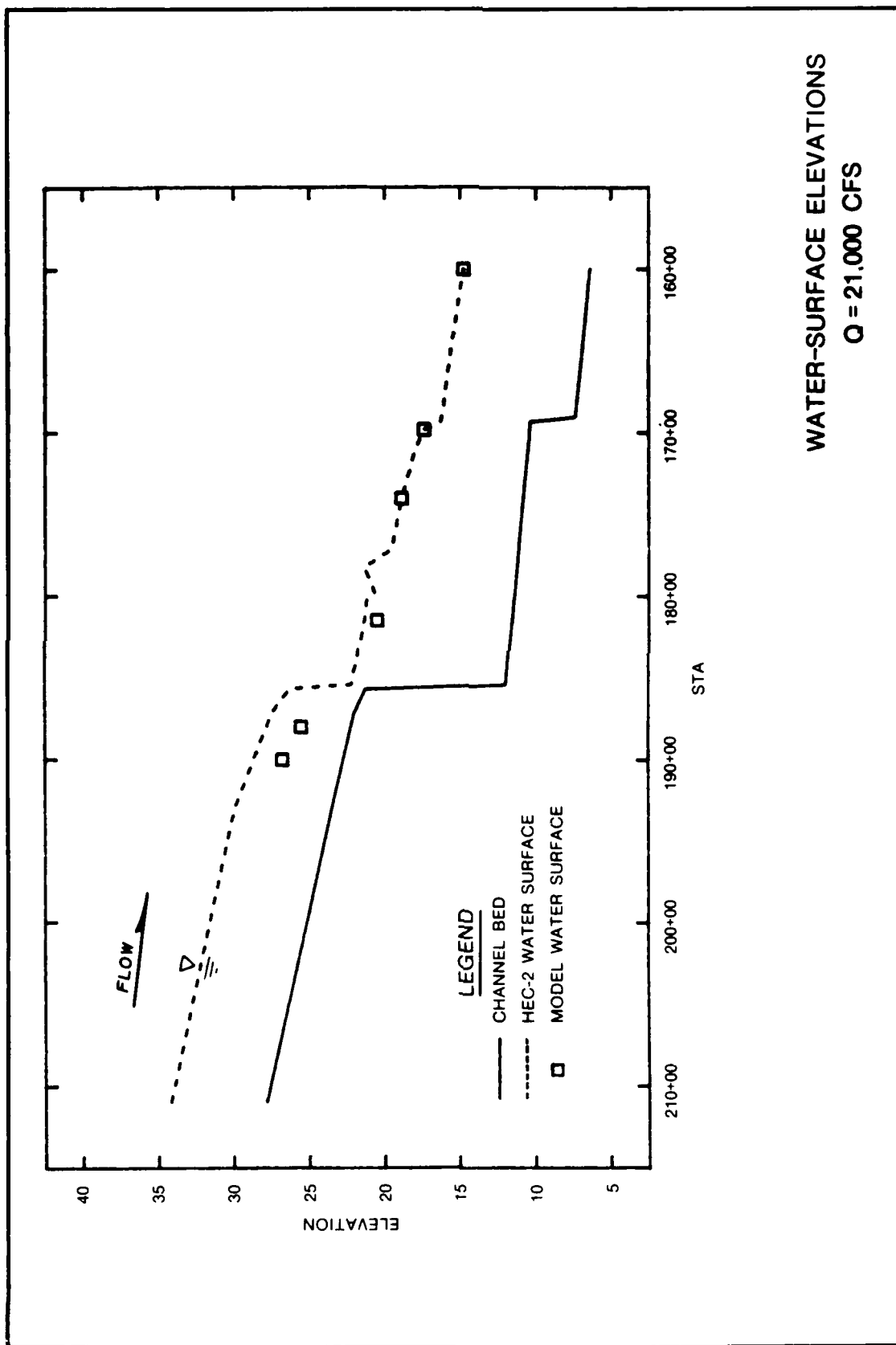
b. $Q = 45,000$ cfs



c. $Q = 60,000$ cfs

Photo 17. Flow conditions downstream of the drop structure with the sediment wedge in place





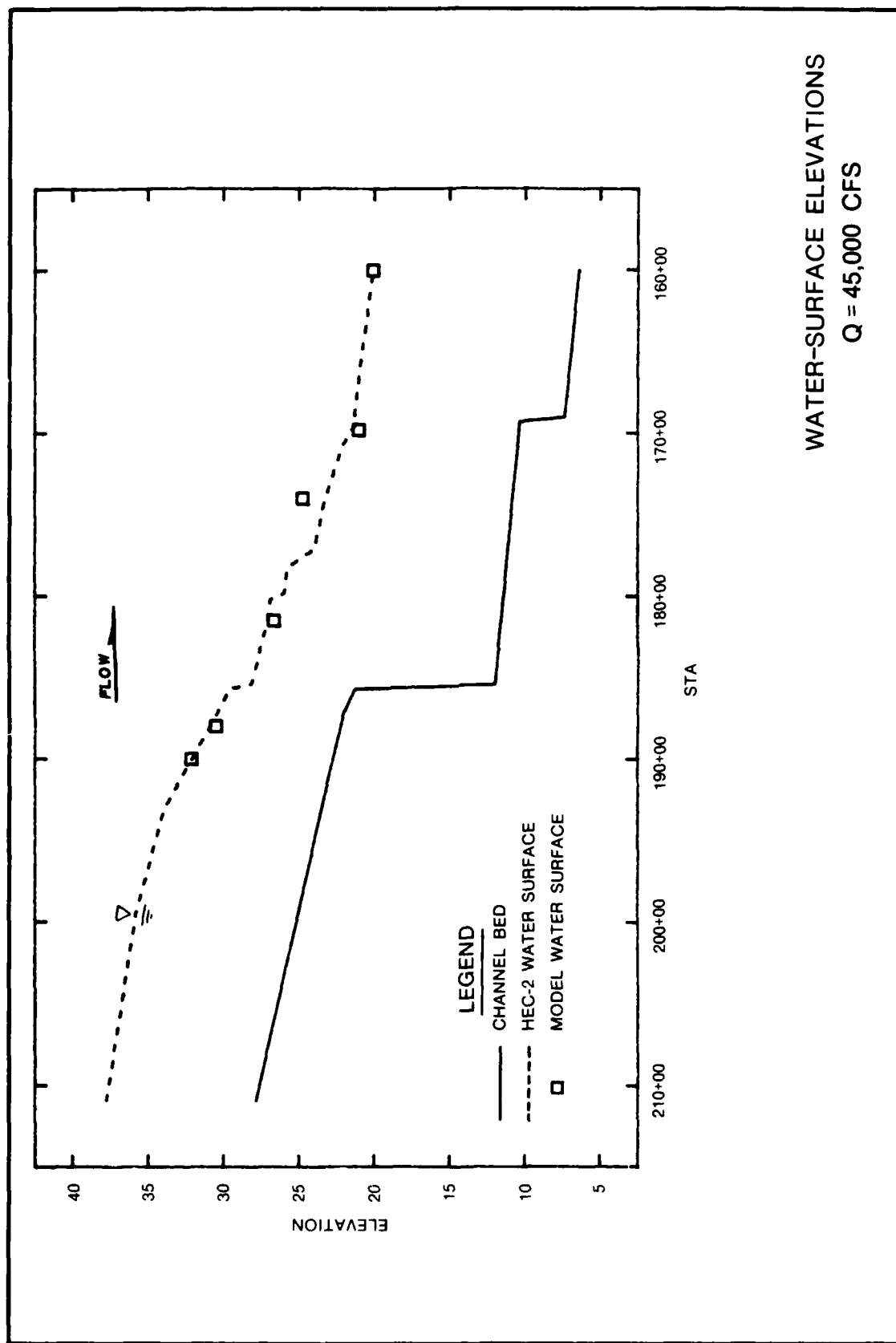
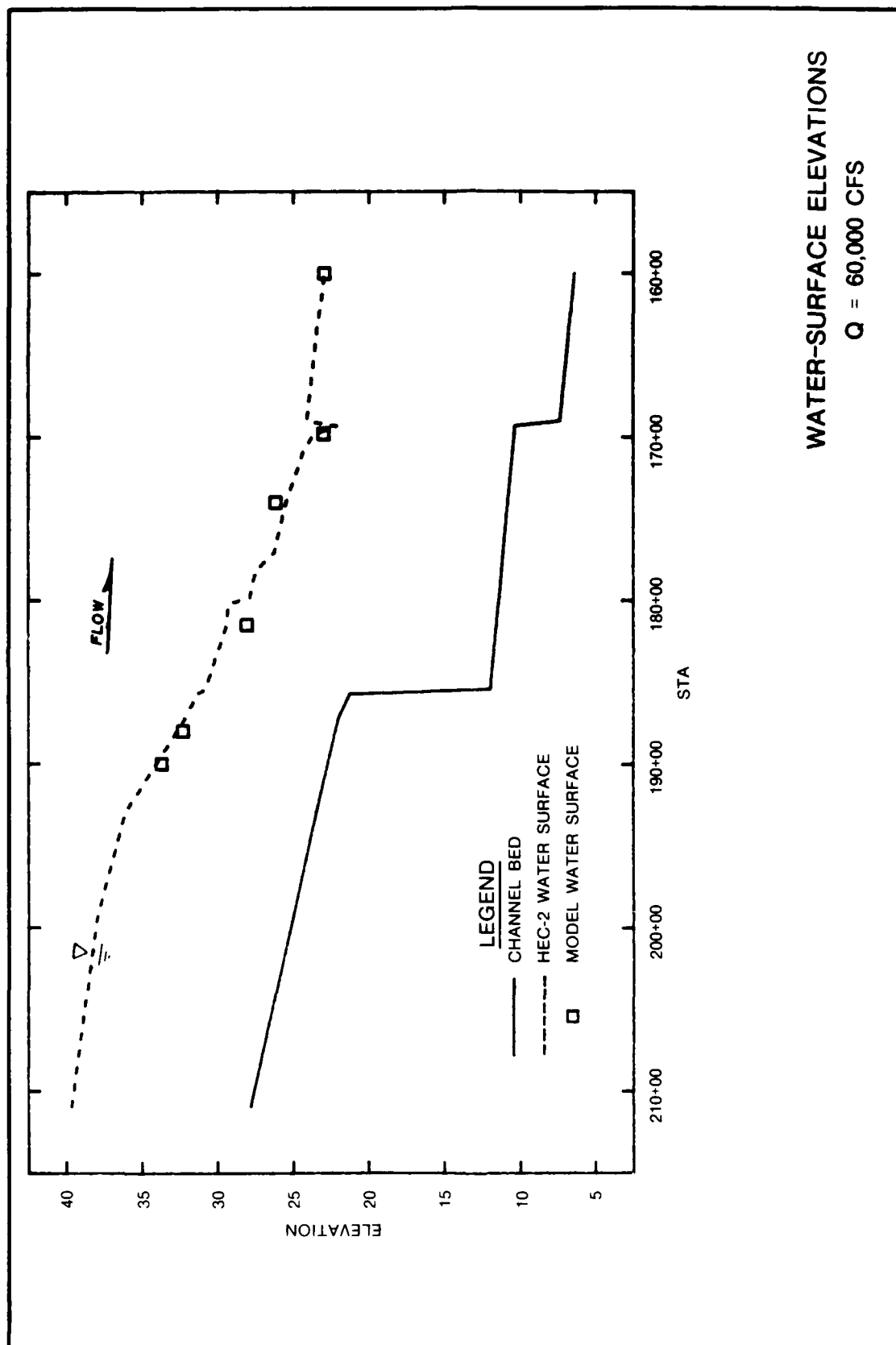
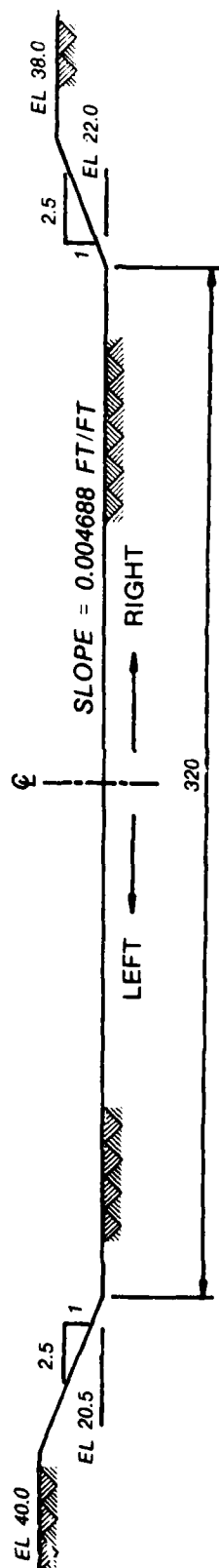
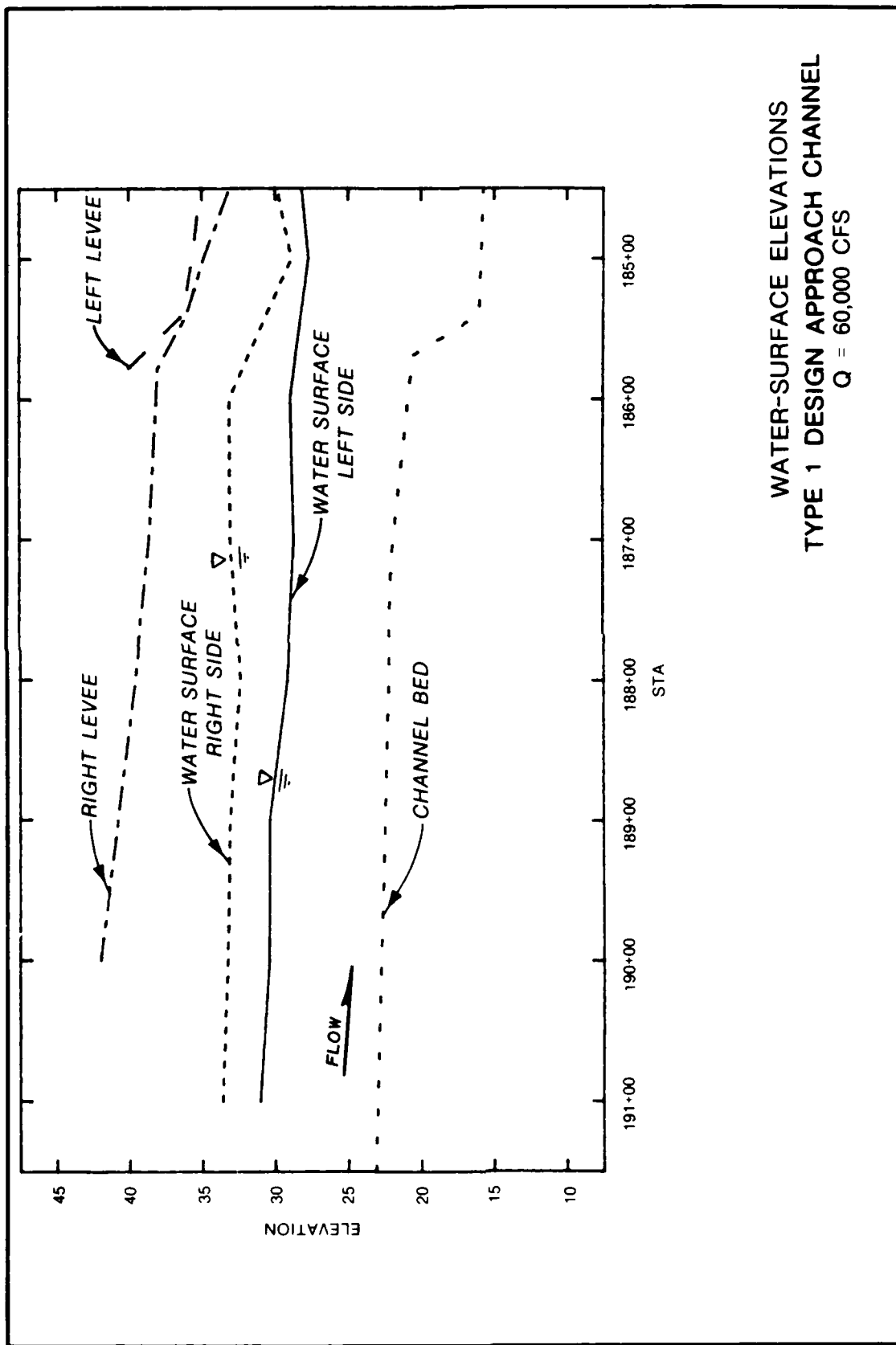


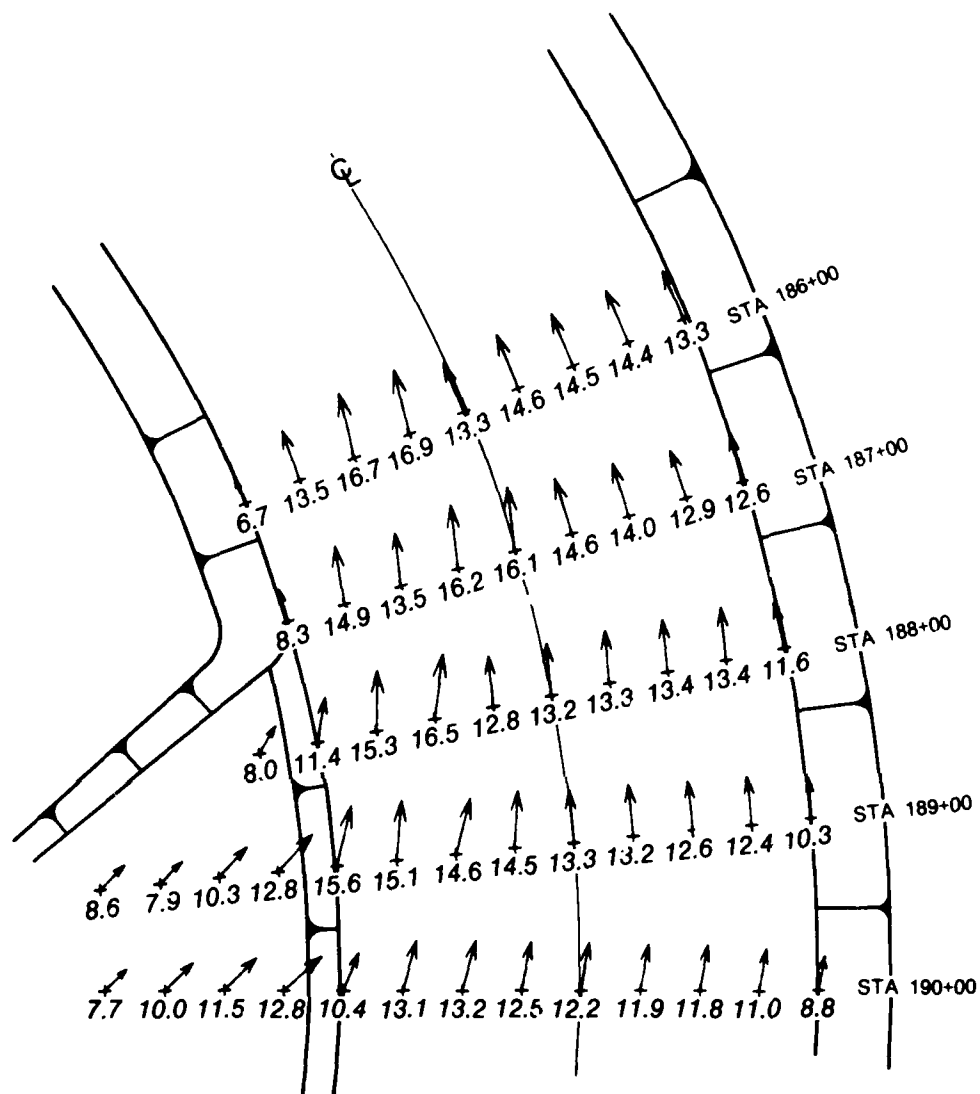
PLATE 3



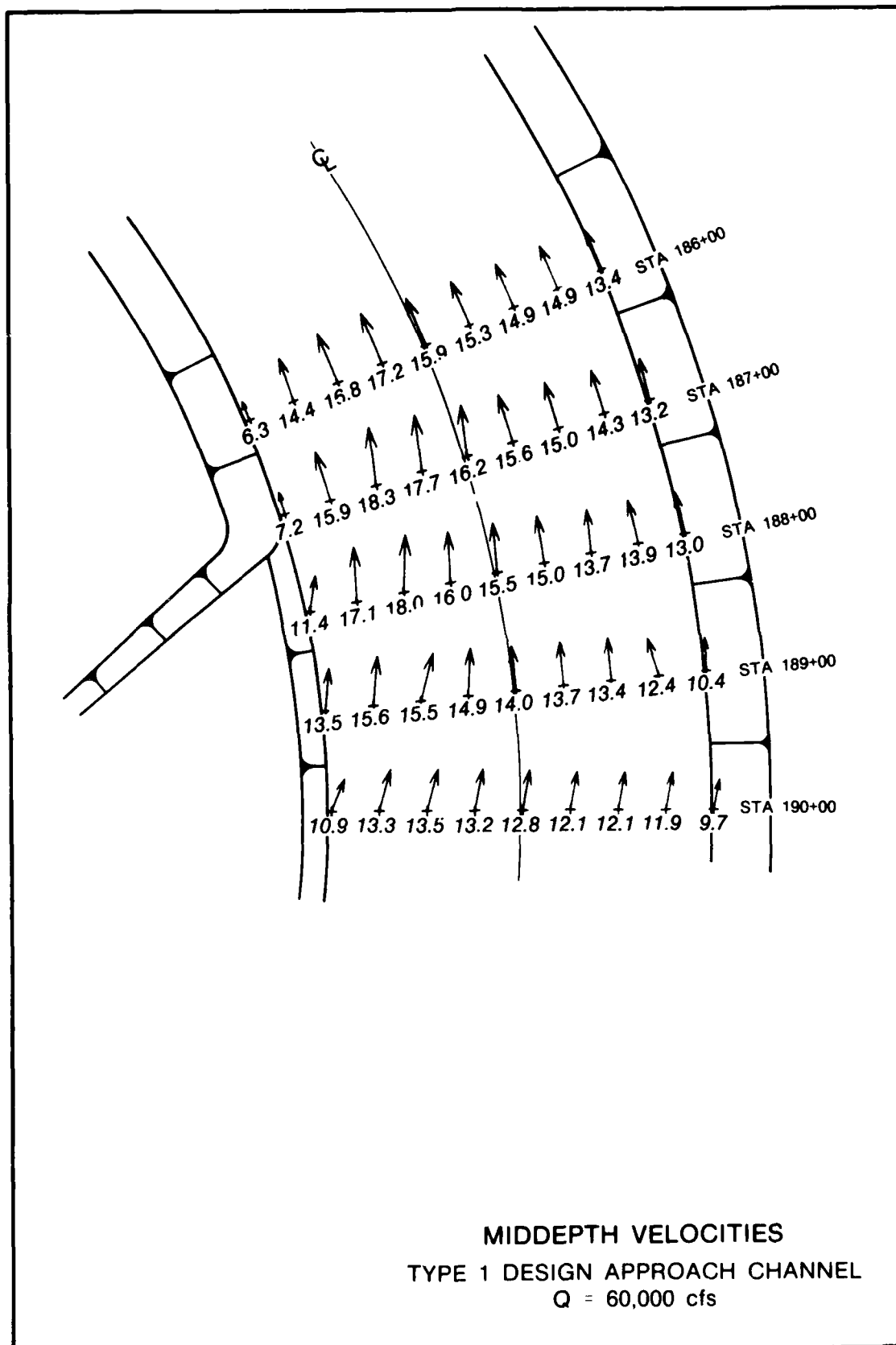


CREST PROFILE LOOKING DOWNSTREAM
STA 185+ 43.65

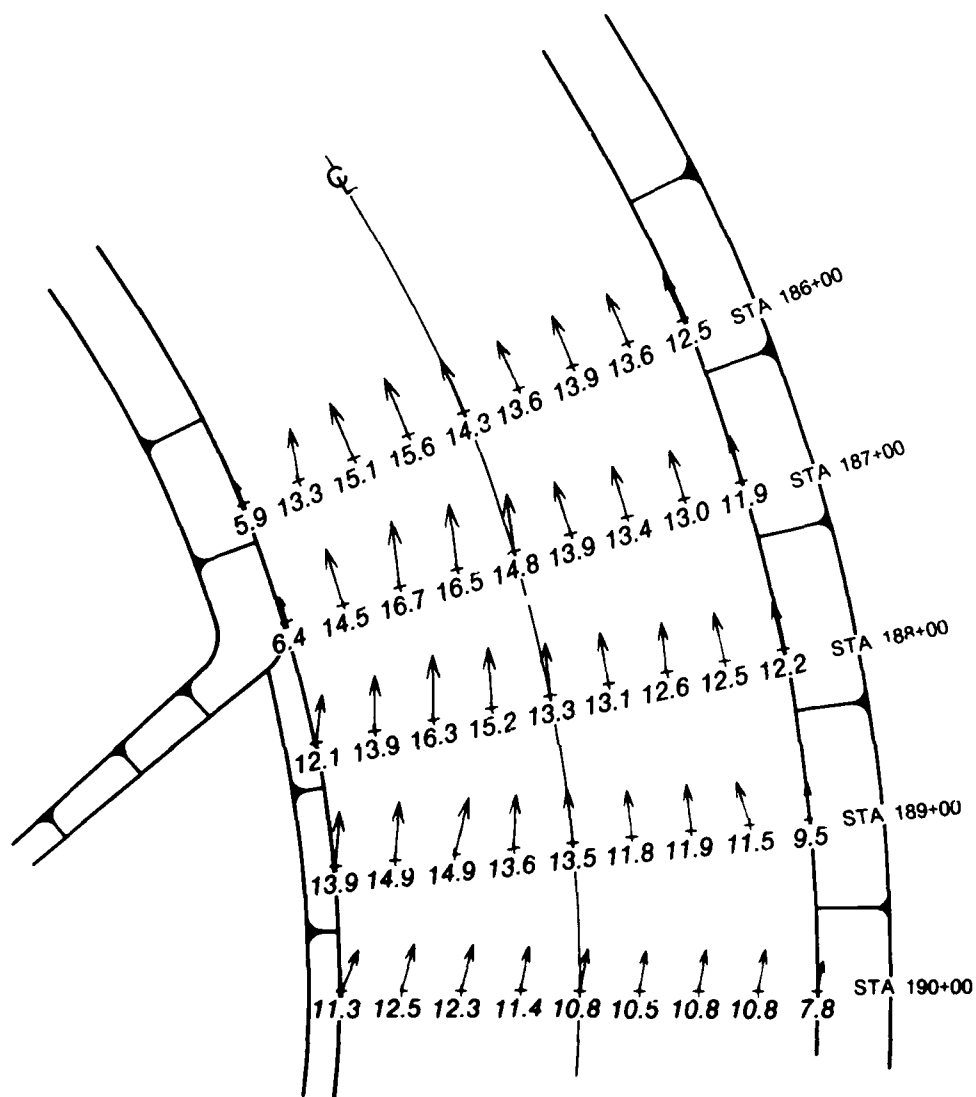




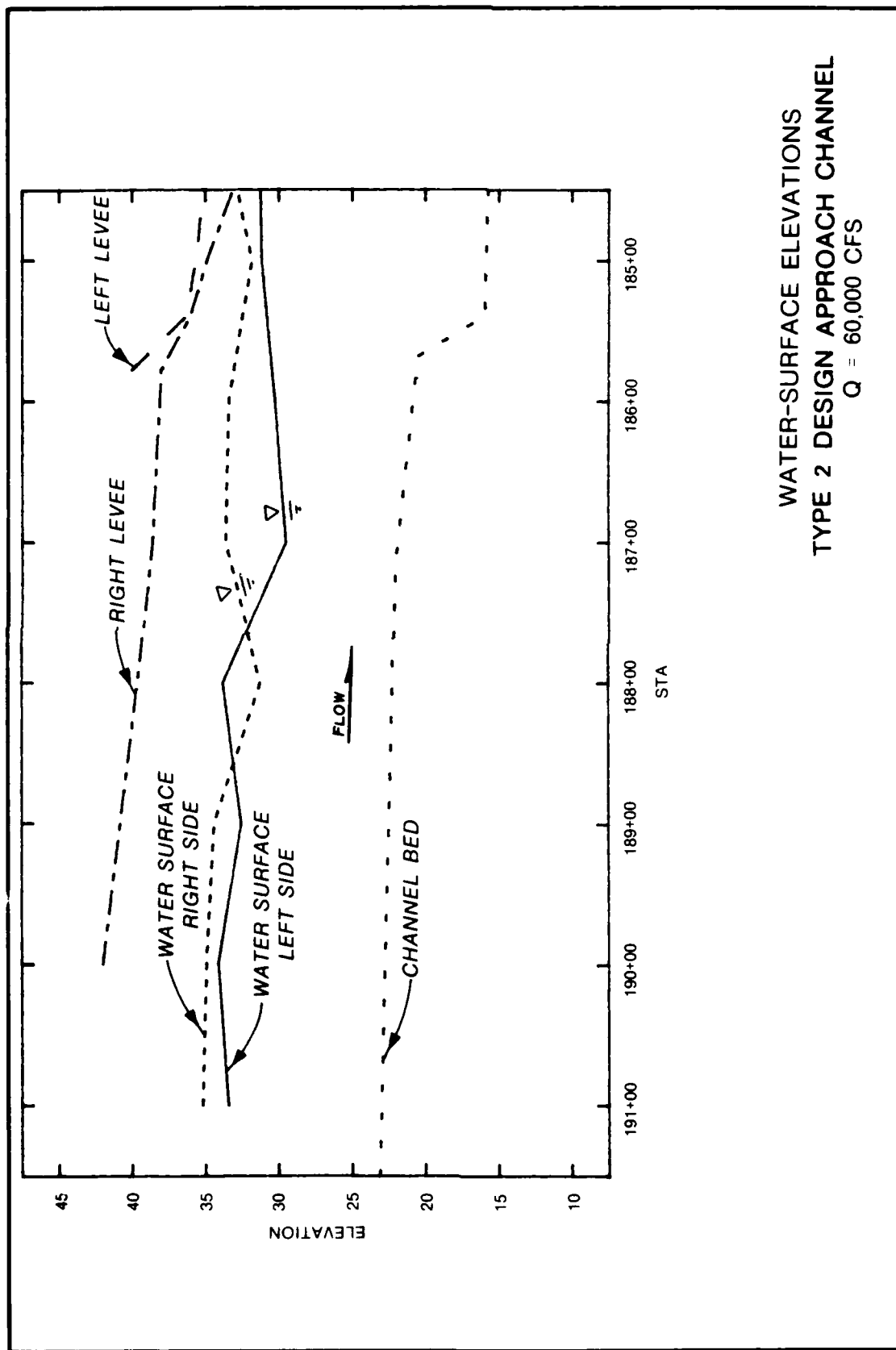
SURFACE VELOCITIES
 TYPE 1 DESIGN APPROACH CHANNEL
 $Q = 60,000$ cfs

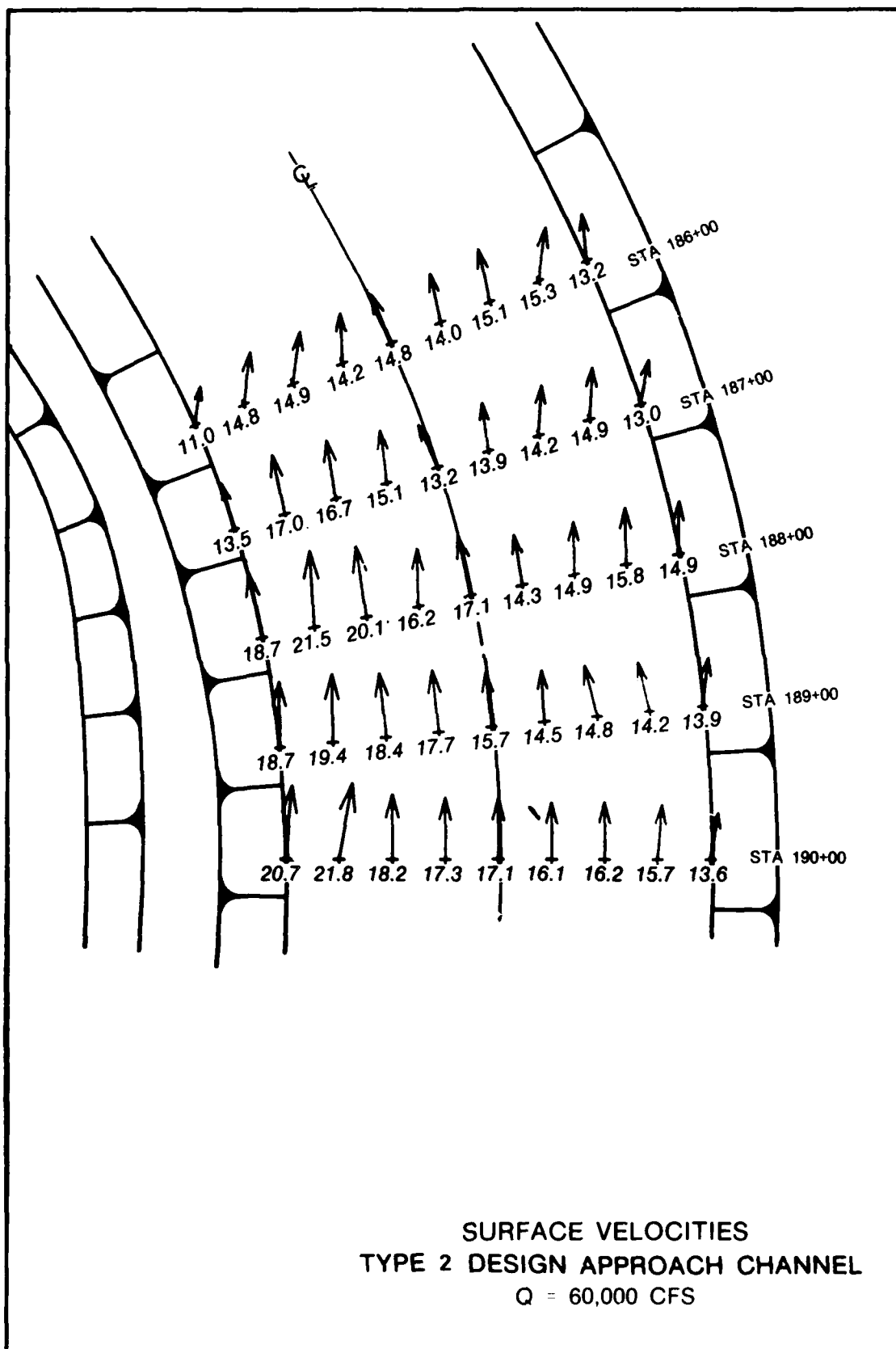


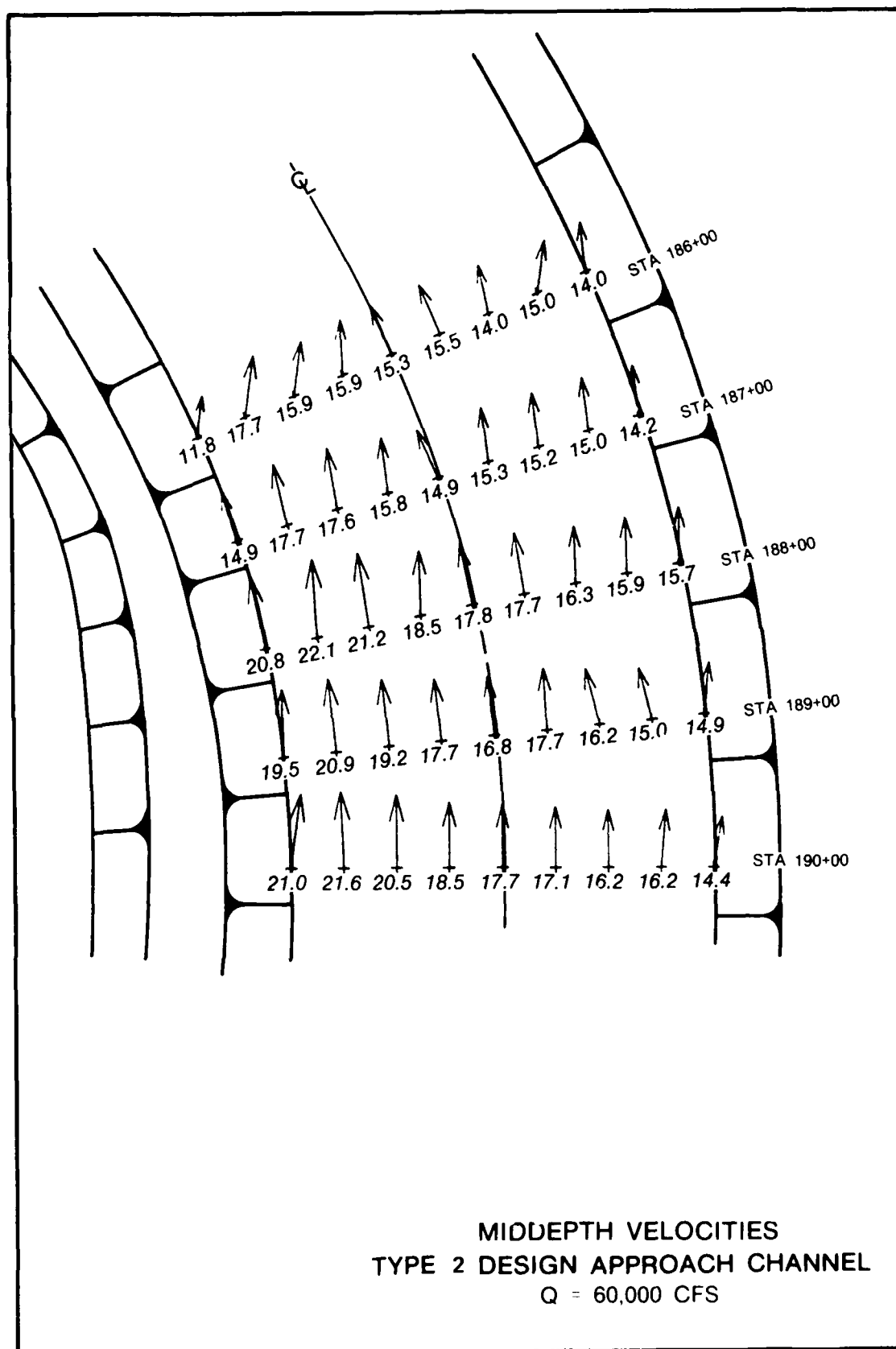
MIDDEPTH VELOCITIES
TYPE 1 DESIGN APPROACH CHANNEL
 $Q = 60,000$ cfs

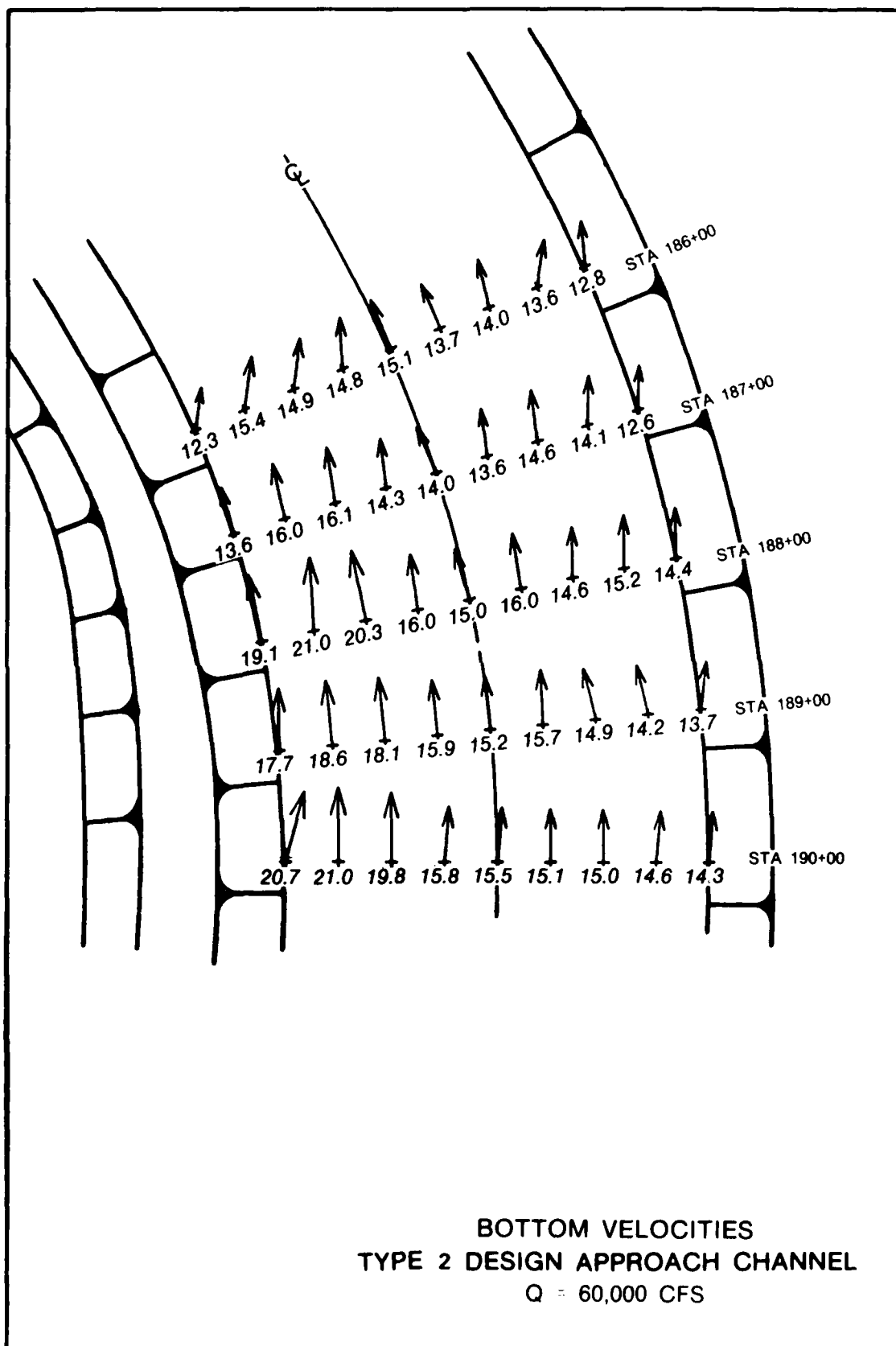


BOTTOM VELOCITIES
 TYPE 1 DESIGN APPROACH CHANNEL
 $Q = 60,000$ cfs

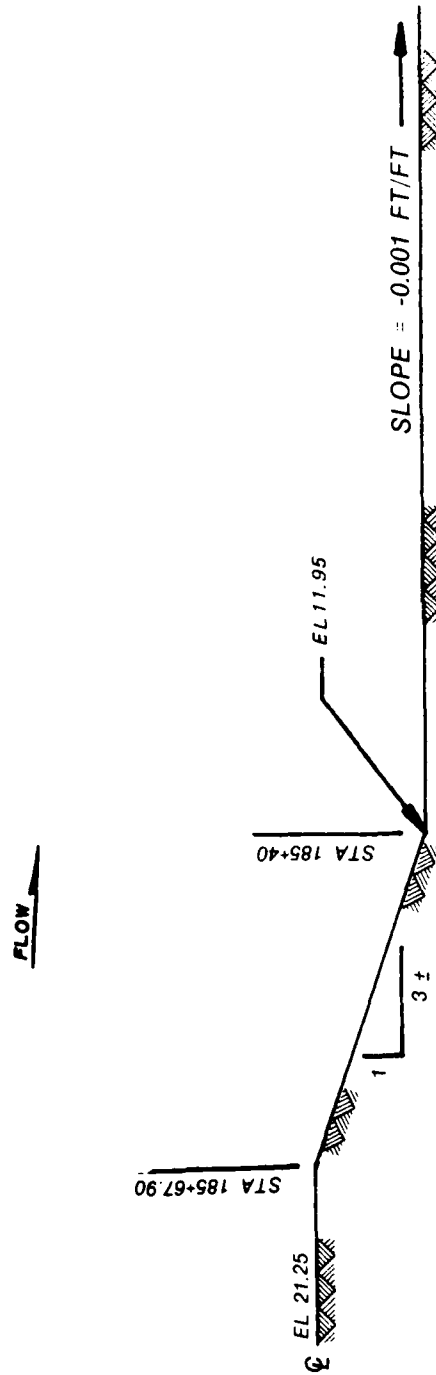


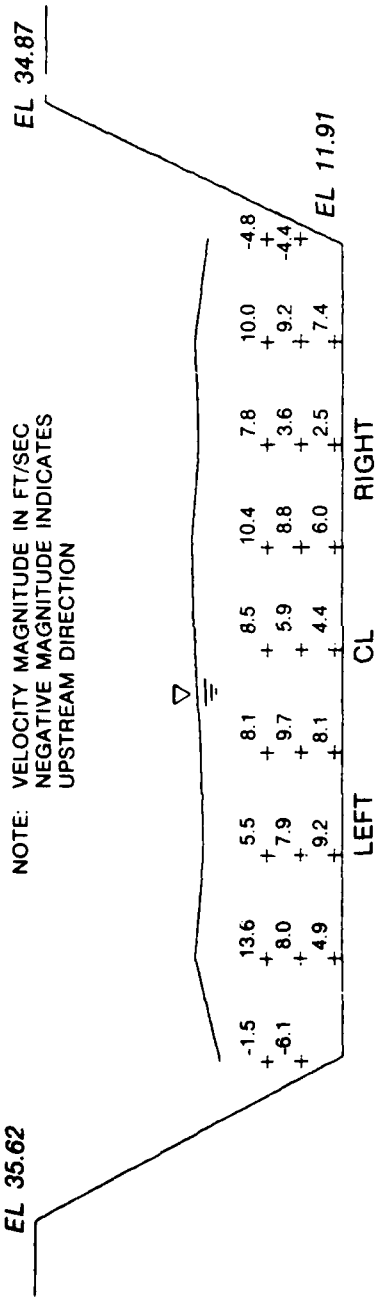




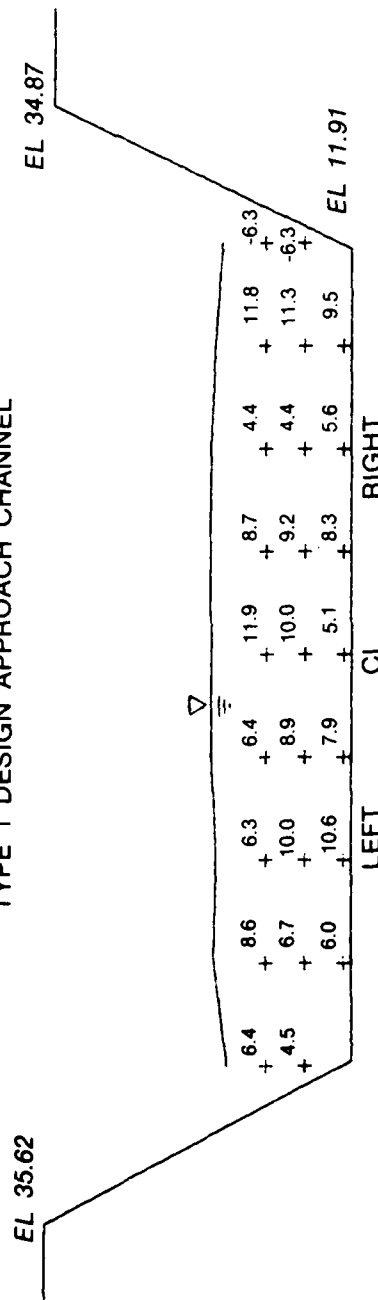


DROP STRUCTURE
TYPE 1 ORIGINAL DESIGN



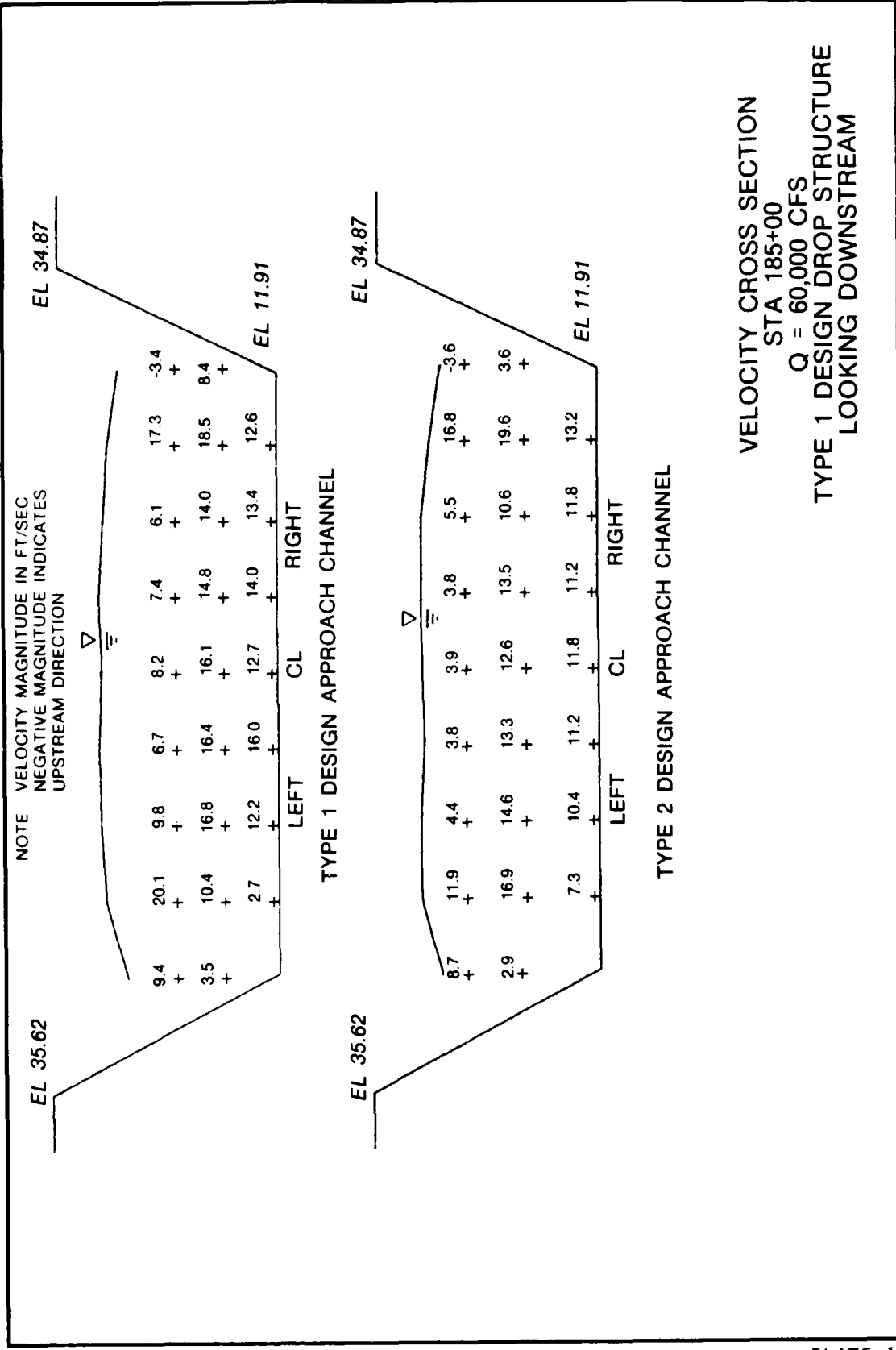


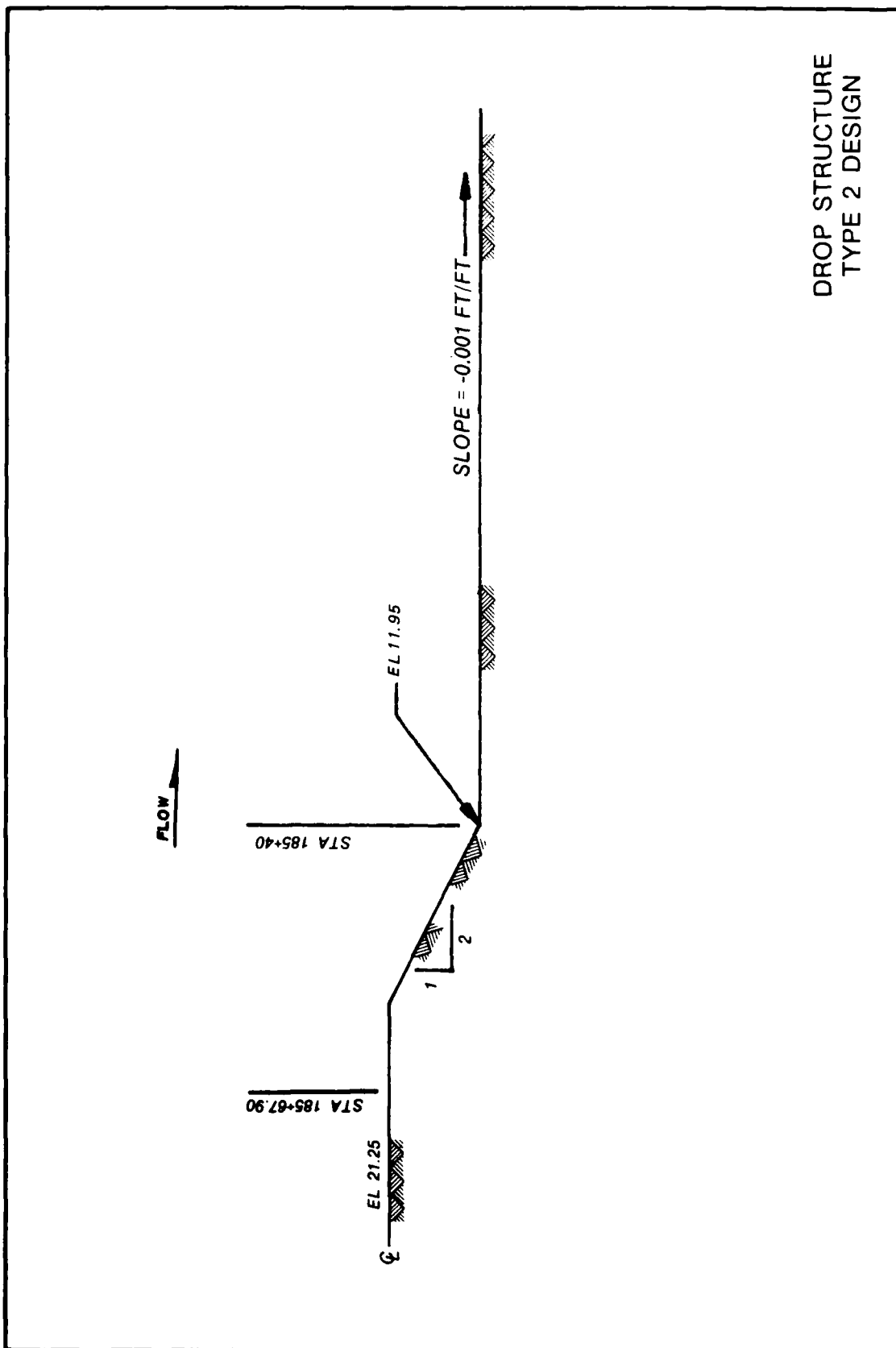
TYPE 1 DESIGN APPROACH CHANNEL

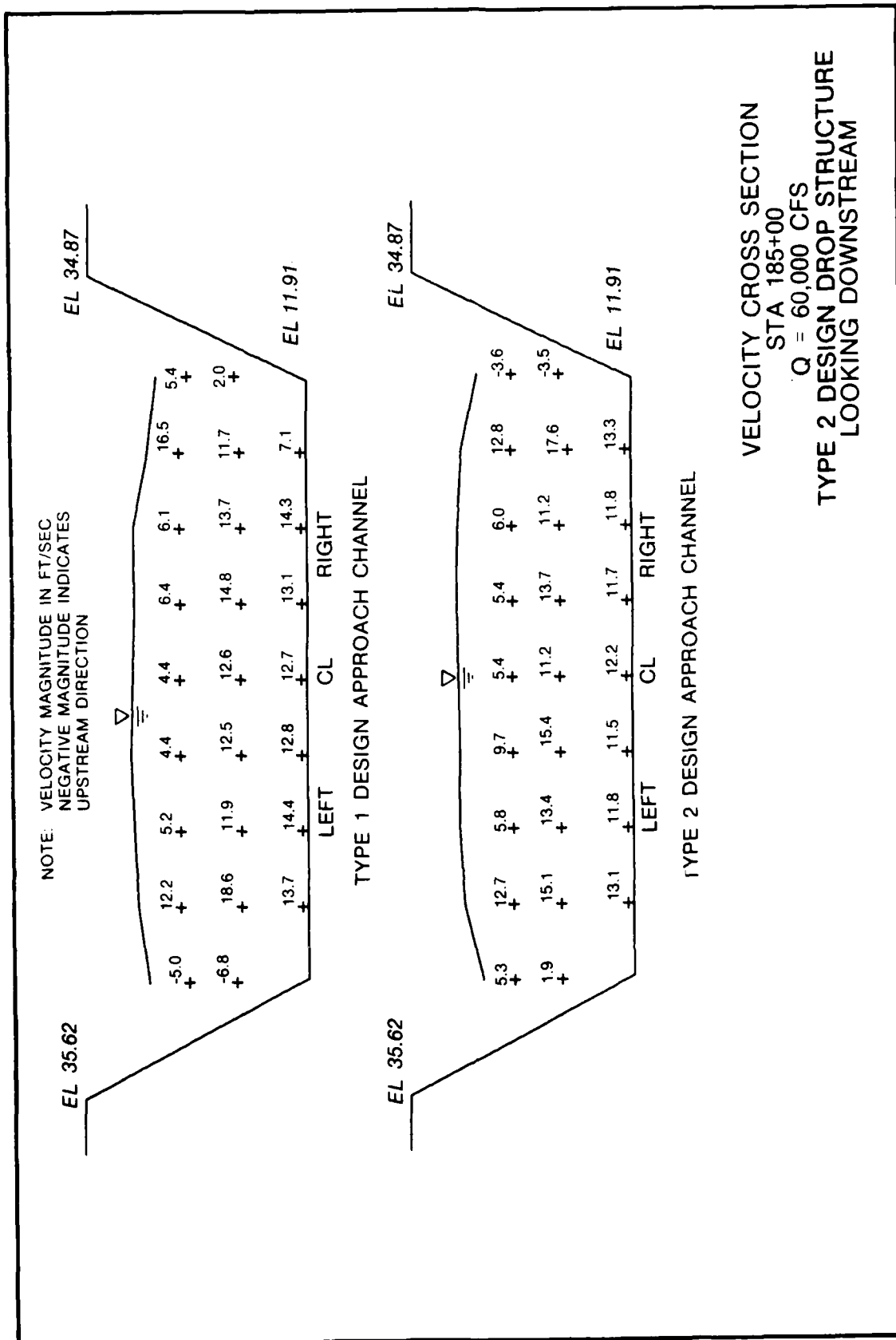


TYPE 2 DESIGN APPROACH CHANNEL

VELOCITY CROSS SECTION
STA 185+00
Q = 21,000 CFS
TYPE 1 DESIGN DROP STRUCTURE
LOOKING DOWNSTREAM







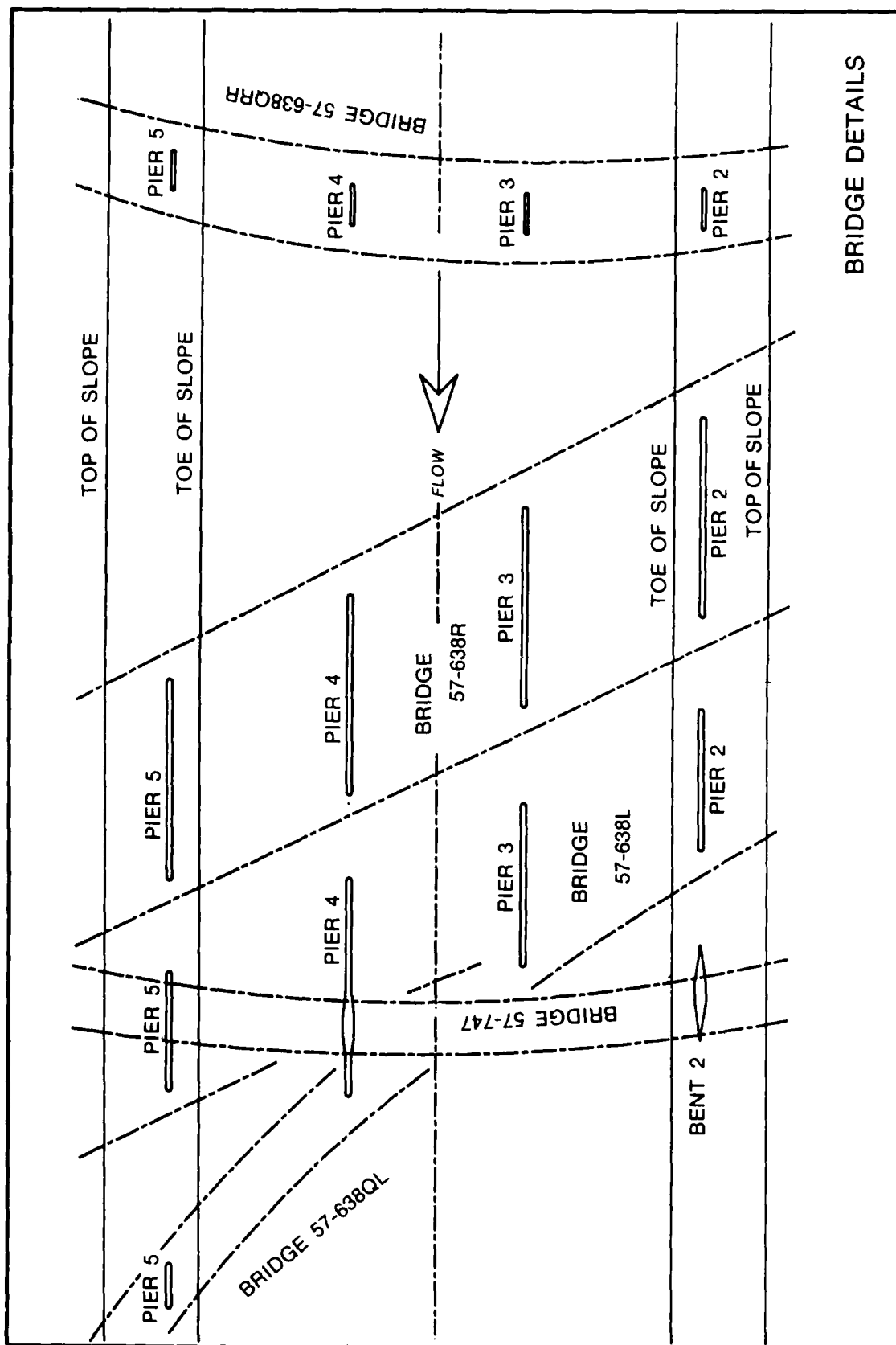


PLATE 19

26.58	25.74	26.54	30.42
+	+	+	+
26.70	57-638QRR PIER 5		30.94
+			+
26.86	+ 26.54	+ 25.94	30.74
+			+

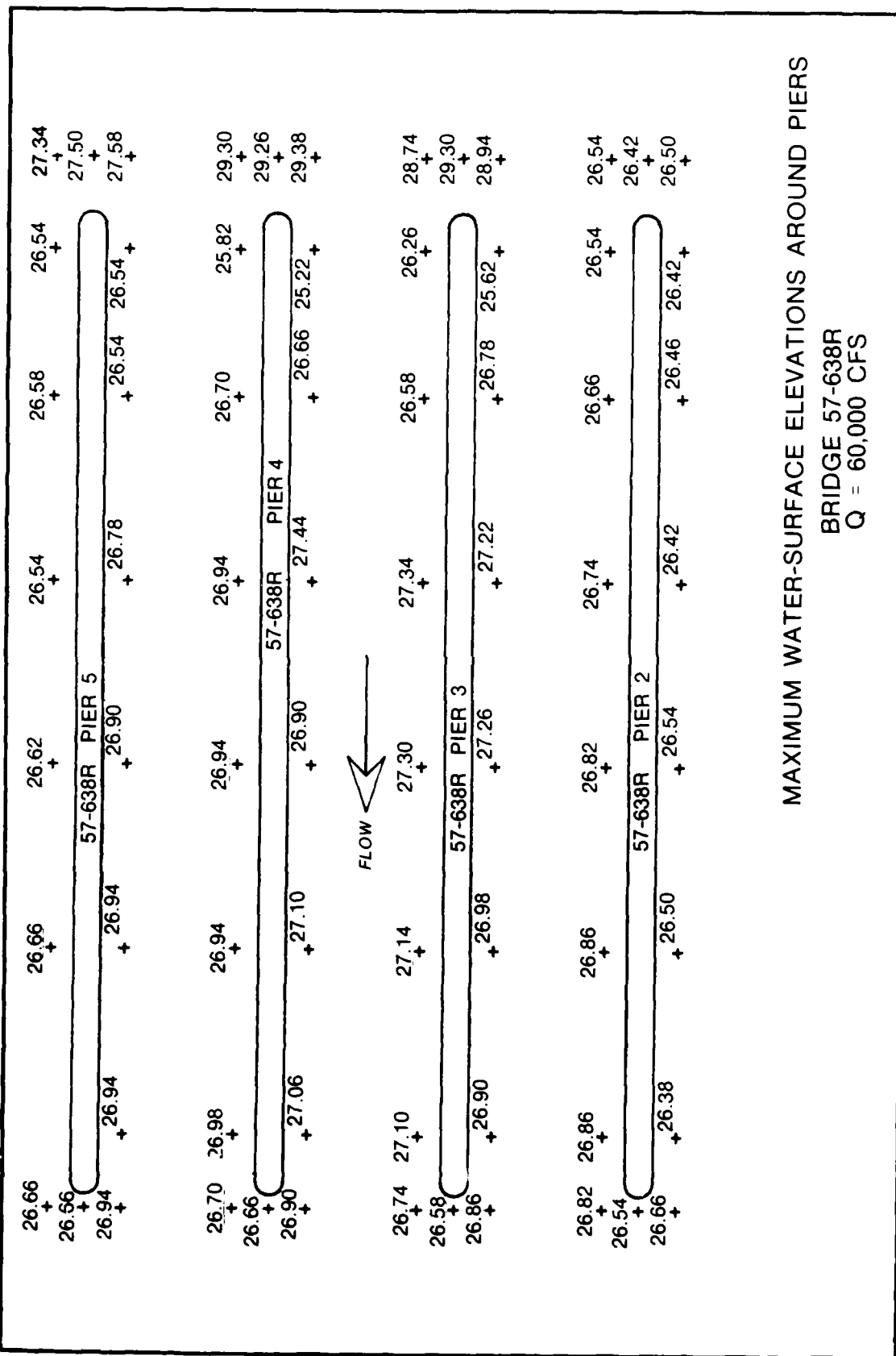
25.70	25.22	23.86	29.86
+	+	+	+
25.62	57-638QRR PIER 4		29.70
+			+
25.70	+ 25.02	+ 26.02	30.18
+			+

FLOW ←

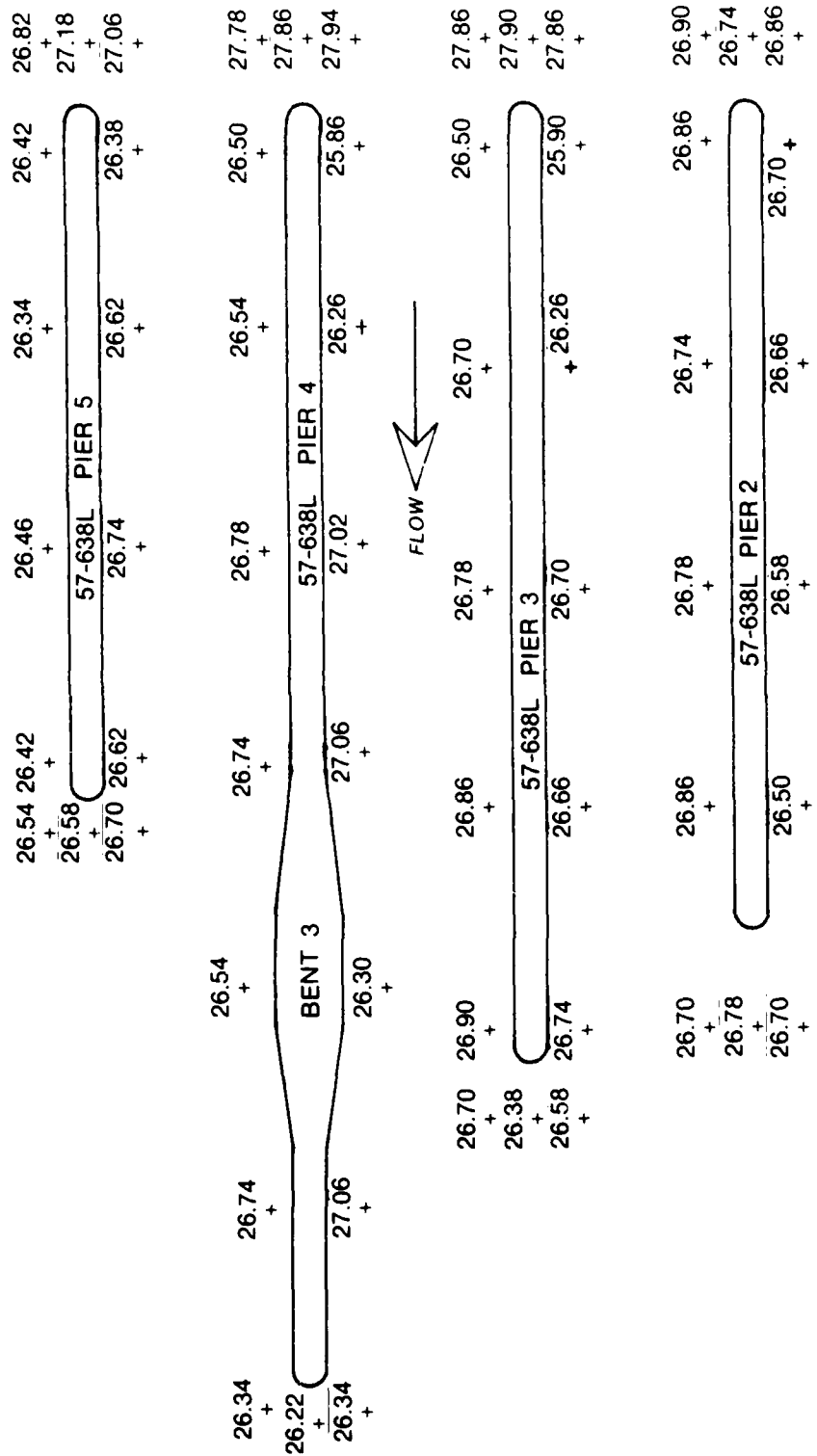
25.82	25.38	23.62	29.70
+	+	+	+
25.78	57-638QRR PIER 3		29.82
+			+
25.50	+ 24.70	+ 24.98	30.02
+			+

26.06	26.10	26.14	28.90
+	+	+	+
25.98	57-638QRR PIER 2		29.30
+			+
26.02	+ 25.82	+ 25.78	28.90
+			+

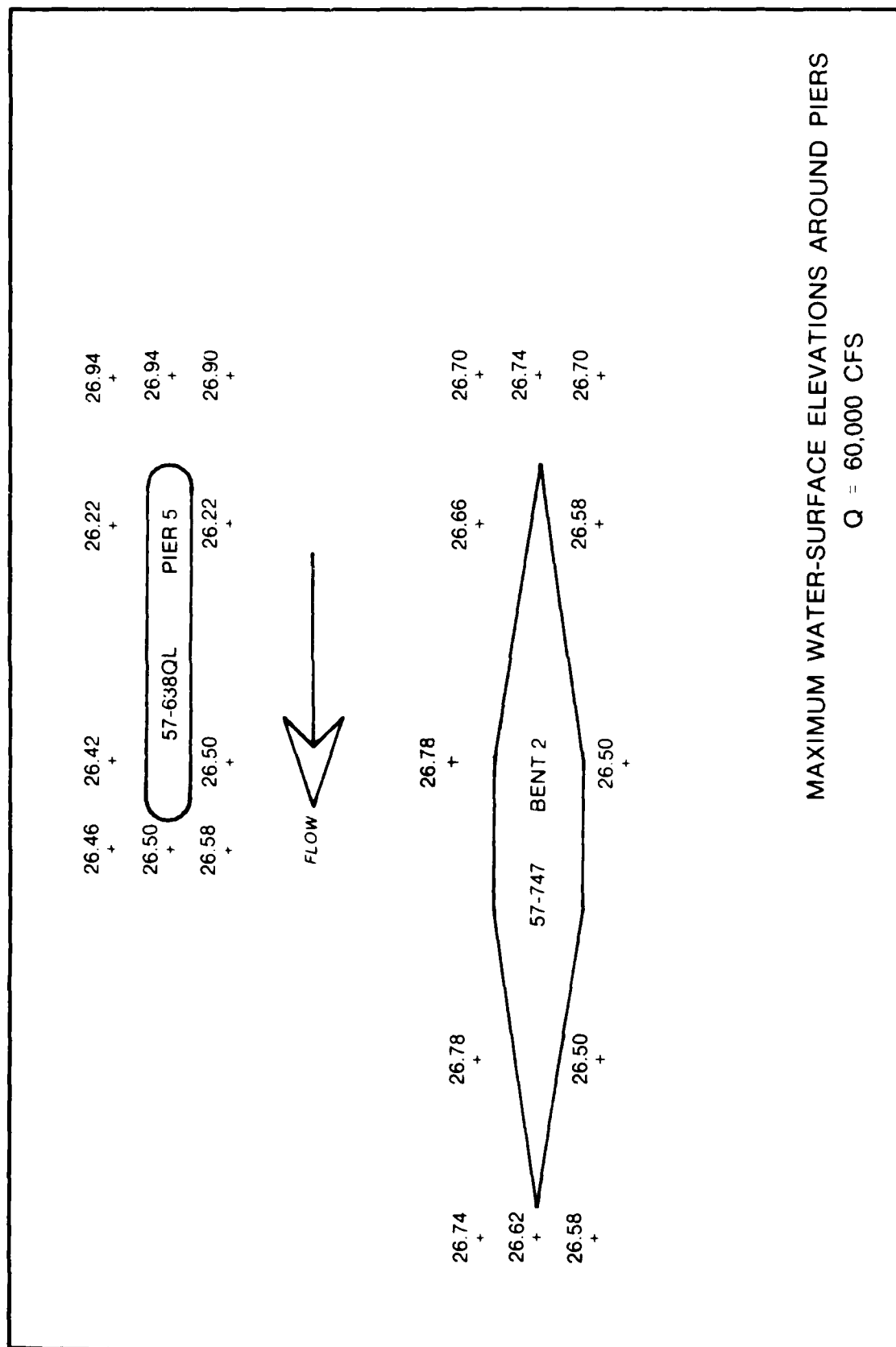
MAXIMUM WATER-SURFACE ELEVATIONS AROUND PIERS
 BRIDGE 57-638QRR
 Q = 60,000 CFS

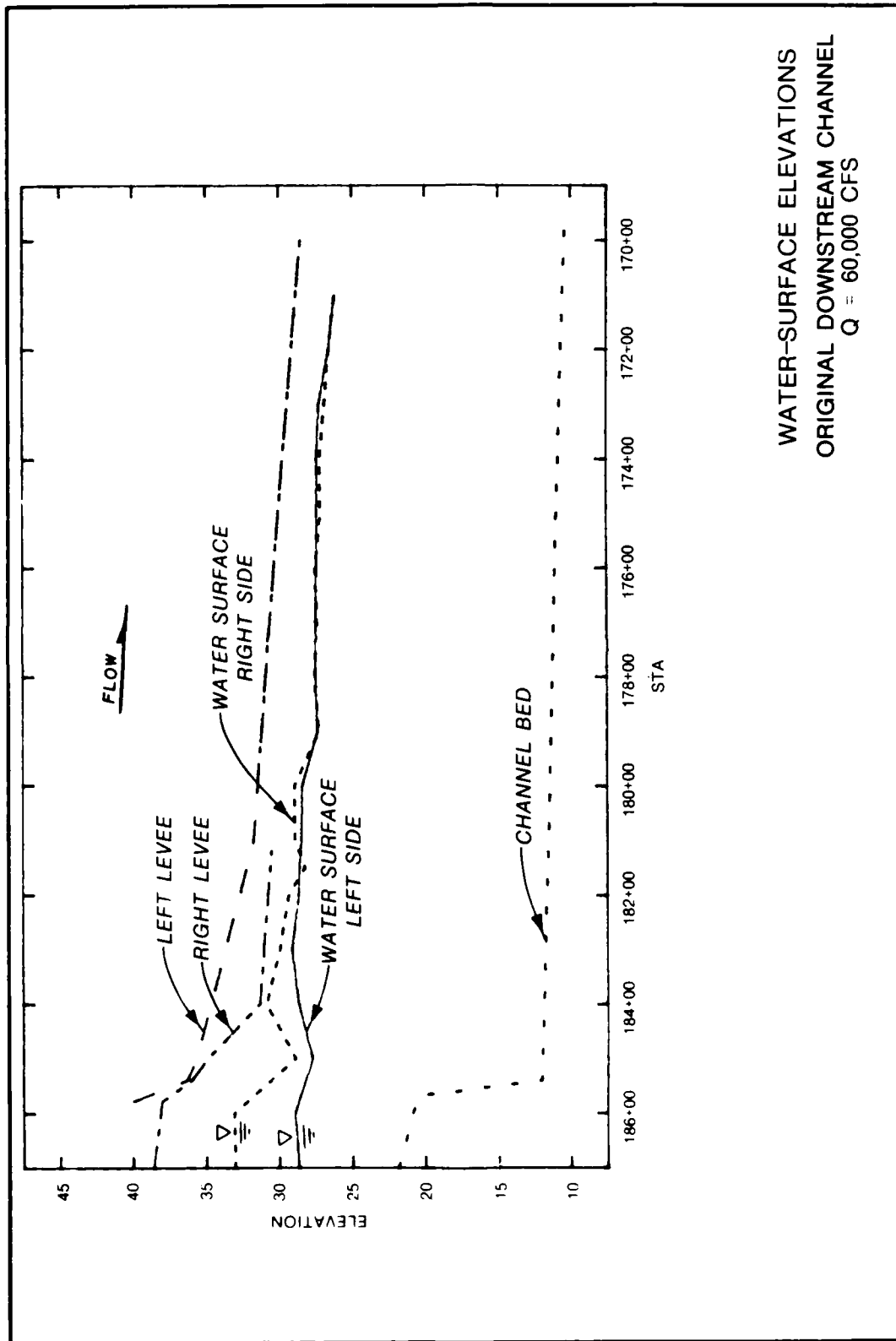


MAXIMUM WATER-SURFACE ELEVATIONS AROUND PIERS
BRIDGE 57-638R
Q = 60,000 CFS



MAXIMUM WATER-SURFACE ELEVATIONS AROUND PIERS
BRIDGE 57-638L
Q = 60,000 CFS





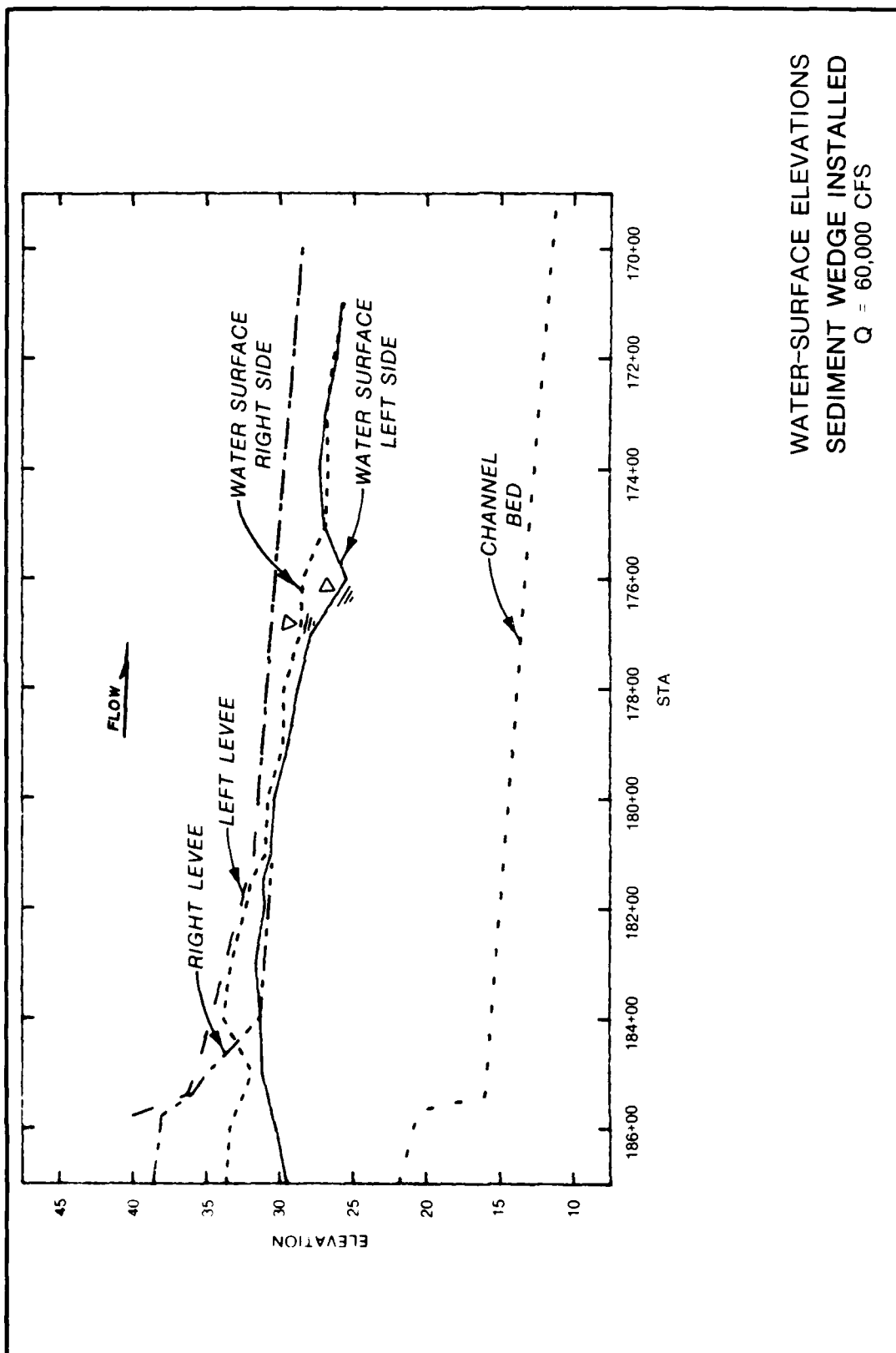


PLATE 25

29.70	29.62	29.58	31.82
+	+	+	+
29.74	57-638QRR PIER 5		32.18
+			+
30.02	29.10	29.46	32.46
+	+	+	+

28.90	29.14	28.02	32.78
+	+	+	+
29.02	57-638QRR PIER 4		33.50
+			+
29.42	29.38	28.50	32.78
+	+	+	+

FLOW ←

28.66	27.70	28.70	33.02
+	+	+	+
28.38	57-638QRR PIER 3		33.58
+			+
28.70	28.38	29.98	33.30
+	+	+	+

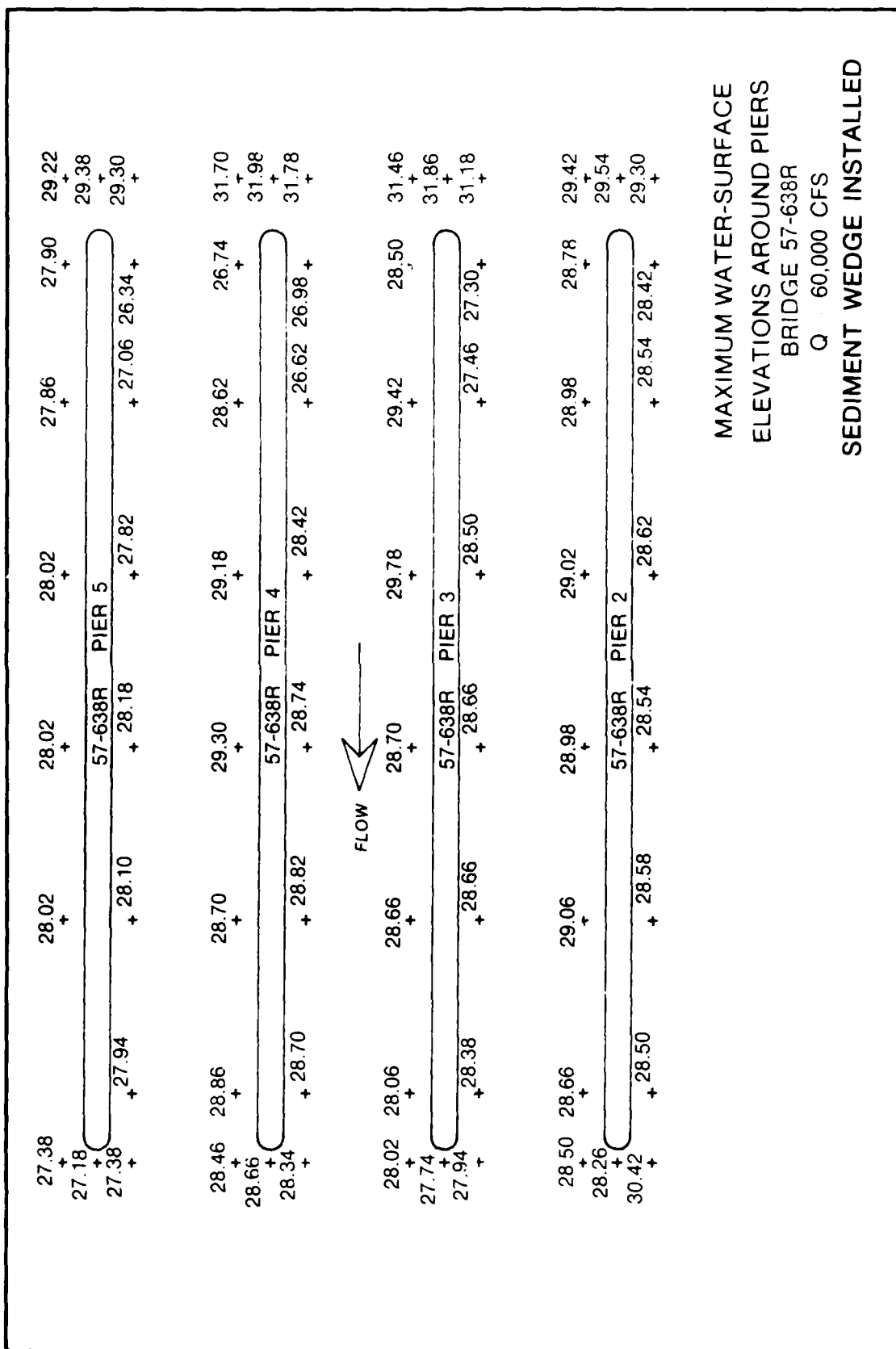
28.22	28.26	28.38	30.90
+	+	+	+
28.14	57-638QRR PIER 2		30.98
+			+
28.30	28.18	28.02	30.90
+	+	+	+

MAXIMUM WATER-SURFACE ELEVATIONS AROUND PIERS

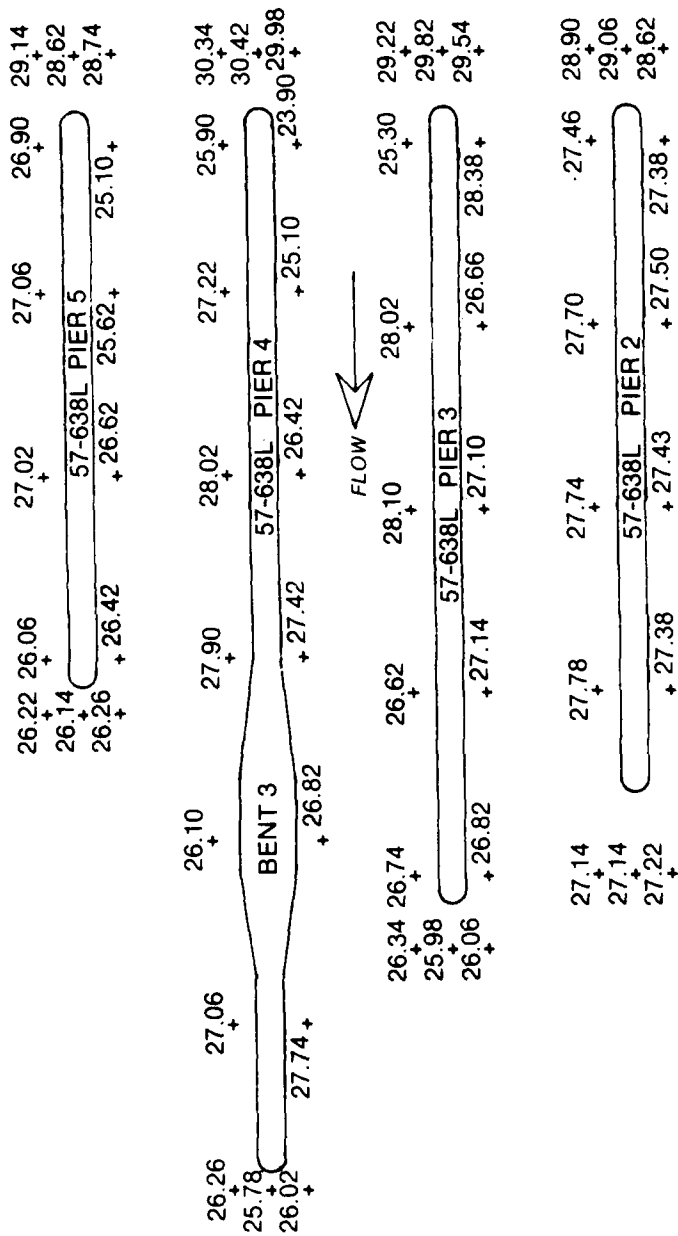
BRIDGE 57-638QRR

Q = 60,000 CFS

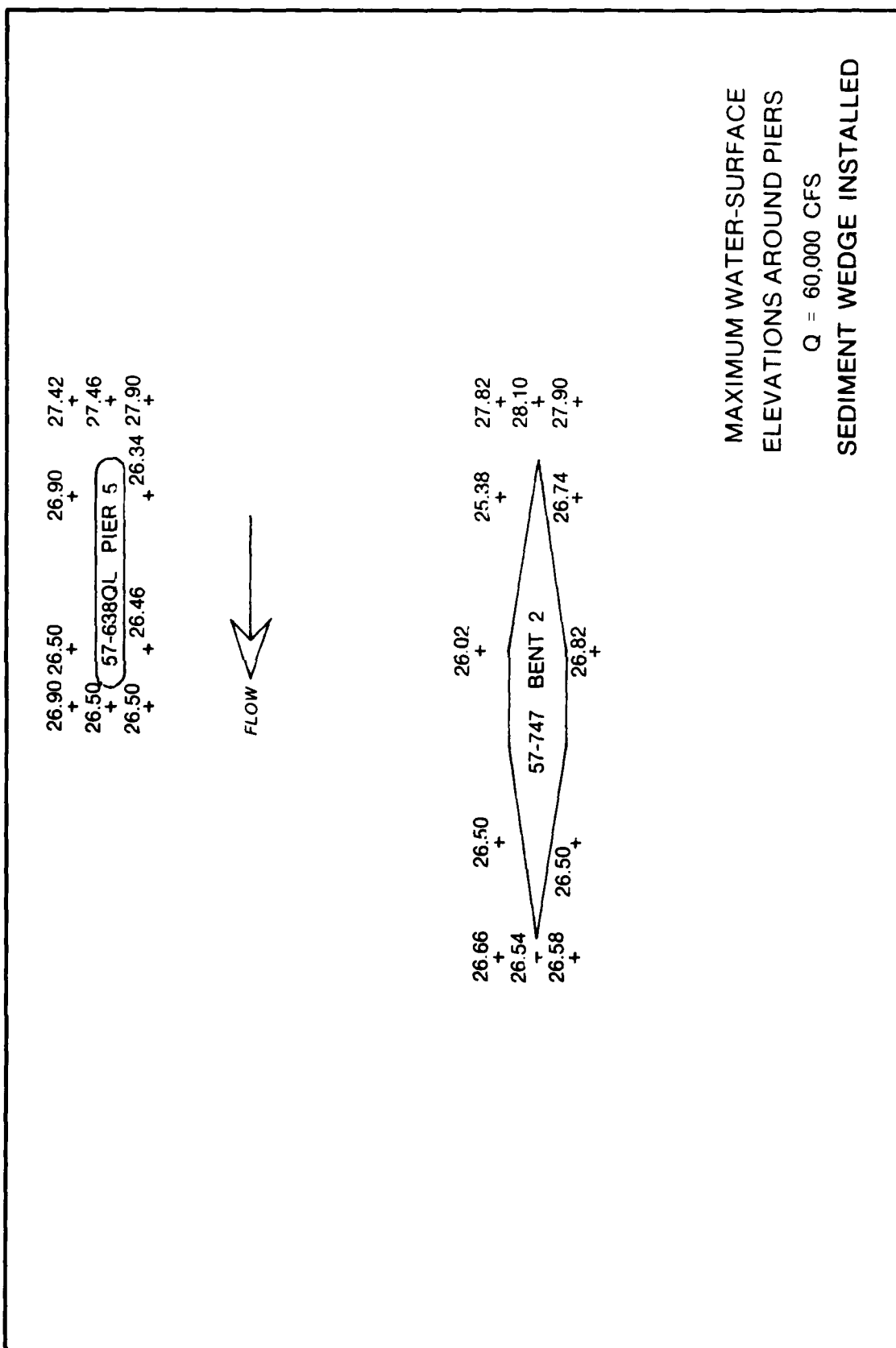
SEDIMENT WEDGE INSTALLED



MAXIMUM WATER-SURFACE
ELEVATIONS AROUND PIERS
BRIDGE 57-638R
Q = 60,000 CFS
SEDIMENT WEDGE INSTALLED



MAXIMUM WATER-SURFACE
ELEVATIONS AROUND PIERS
BRIDGE 57-638L
Q = 60,000 CFS
SEDIMENT WEDGE INSTALLED



END

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